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PB-285 663

Conventional and Advanced Sewer Design Concepts for Dual Purpose Flood and Pollution Control A Preliminary Case Study. Elizabeth, New Jersey

Clinton Bogert Associates, Fort Lee, N.J.

Prepared for

Municipal Environmental Research Lab., Cincinnati, Ohio

May 78

United States
Environmental Protection
Agency

Municipal Environmental Research
Laboratory
Cincinnati OH 45268

EPA-600/2-78-090
May 1978

Research and Development

PB 285 663

EPA

Conventional and Advanced Sewer Design Concepts for Dual Purpose Flood and Pollution Control

A Preliminary Case Study, Elizabeth, NJ

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TECHNICAL REPORT DATA

(Please read Instructions on the reverse before completing)

1. REPORT NO.
EPA-600/2-78-090

2.

3. RECIPIENT'S ACCESSION NO.
PB285663

4. TITLE AND SUBTITLE
CONVENTIONAL AND ADVANCED SEWER DESIGN CONCEPTS
FOR DUAL PURPOSE FLOOD AND POLLUTION CONTROL
A Preliminary Case Study, Elizabeth, New Jersey

5. REPORT DATE
May 1978 (Issuing Date)

6. PERFORMING ORGANIZATION CODE

7. AUTHOR(S)
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8. PERFORMING ORGANIZATION REPORT NO.

9. PERFORMING ORGANIZATION NAME AND ADDRESS
Clinton Bogert Associates
2125 Center Avenue
Fort Lee, New Jersey 07024

10. PROGRAM ELEMENT NO.
1BC611

11. ~~CONTRACT~~/GRANT NO.
S-802971

12. SPONSORING AGENCY NAME AND ADDRESS
Municipal Environmental Research Laboratory--Cin.,OH
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

13. TYPE OF REPORT AND PERIOD COVERED
Final

14. SPONSORING AGENCY CODE
EPA/600/14

15. SUPPLEMENTARY NOTES
Project Officer: Anthony N. Tafuri, Phone: 201-321-6679

16. ABSTRACT

Alternatives for pollution abatement from combined sewer overflows and stormwater discharges were evaluated. Separate storm and sanitary, conventional combined, and advanced combined systems with varying amounts of in-pipe and/or satellite storage and controlled flow routing were compared. Cost-effectiveness assuming a desired effluent quality and new sewer system was determined. The effects on pollution abatement and cost of changing various elements (collection system, interceptors, storage and treatment works) of the system were investigated. SWMM and STORM were employed to design sewers, analyze the quantity and quality of combined sewage and stormwater runoff, and analyze a continuous 12-year, real rainfall record. The overflow frequency, pollutants, and volume for 59 alternatives were determined. The runoff and overflow characteristics developed for 265 hectares, were used to determine the characteristics from the remaining 1785 hectares to plan a citywide sewer system. The study evaluated the effects of (a) differing rainfall patterns and intensities on pollutant concentrations, (b) varying interceptor and treatment capacity on pollutants discharged, and (c) peak flow equalizing for different levels of treatment. The long-term pollutional loads from combined sewage overflows were quantified as well as the effect on overflow control of varying the amount and location of storage.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
*Rainfall, *Runoff, *Storm sewers, *Combined sewer, *Sanitary sewers, *Overflows, *Hydraulics, Hydrology, *Cost effectiveness, *Mathematical models, Computer programs, Methodology, *Surface water runoff, *Water pollution, Water quality, Water storage	Combined sewer overflow management, In-system storage, In-pipe storage, Satellite storage, Off-line storage, Hydrographs, Pollutographs, Frequency analysis	13B

18. DISTRIBUTION STATEMENT
Release to Public

19. SECURITY CLASS *(This Report)*
Unclassified

20. SECURITY CLASS *(This page)*
Unclassified

21. NO. OF PAGES
231

22. PRICE
PCA11 / MF A01

★ U.S. GOVERNMENT PRINTING OFFICE: 1978—757-140/1362

EPA-600/2-78-090
May 1978

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1-A

EPA-600/2-78-090
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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution. It involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources. This work is to facilitate the preservation and treatment of public drinking water supplies and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; part of a most vital communications link between the researcher and the user community.

Alleviation of the deleterious effects of combined sewer overflows and storm water discharges on the nation's waterways depends upon characterization of the pattern, quantity and quality of these flows and evaluation of alternative methods to achieve cost-effective control. This report compares, using the urban area of the City of Elizabeth, New Jersey, the cost-effectiveness of alternatives for a community-wide sewer system. Advanced sewer designs, providing controlled flow routing, are compared with conventional combined and separate sewer systems. The report also describes a practical methodology for planning cost-effective facilities to abate pollution from wet-weather flows.

Francis T. Mayo
Director
Municipal Environmental
Research Laboratory

PREFACE

The study was made specifically for Elizabeth, N.J. Because of funding limitations, some approximate analytical methods were used. These approximate methods were evaluated to insure the validity of the conclusion reached as to the main purpose of the report; i.e., the development of relative cost/benefit relationships for various types of sewer systems under identical inputs. In Step 1 (Section 201) Facility Planning, with adequate funding, more exact methods would be used to determine combined sewage overflow pollutant characteristics and to route flows through the system.

Determination of the preferred method of handling rainfall data was outside the scope of this work. In addition, the discrete rainfall/runoff data required for "fine" temporal and distributional analysis and correlation was not available, nor were the funds for their development. However, the importance of runoff volume, as opposed to runoff rate, to achieve combined sewage overflow pollution abatement was recognized. Hence, the rainfall handling was based on synthetic hyetographs which were designed to approximate the runoff volume for the stated return interval at all time segments, rather than peak runoff rates.

The alternative systems developed using the synthetic hyetographs were tested for pollution abatement benefits using STORM and available hourly real rainfall records for a continuous 12 year period.

Although the rainfall input data may not be based on the preferred determination methodology, it still offered a consistent/convenient means for comparatively evaluating the most cost-effective alternative drainage system for combined sewer overflow pollution control.

The concepts developed have been used and the results accepted for Steps 1, 2 and 3 grants for Trenton, New Jersey. Step 1 planning is also proceeding in Elizabeth, New Jersey, based on the findings of this study.

ABSTRACT

Alternatives for pollution abatement from combined sewer overflows and stormwater discharges were evaluated. The following types of systems were compared: separate storm and sanitary system, conventional combined system, and advanced combined systems with varying amounts of in-pipe and/or satellite storage and controlled flow routing. The cost-effectiveness comparison was based on achieving a desired effluent quality assuming a new sewer system. In addition, the effects on pollution abatement and cost of changing various elements (collection system, interceptors, storage and treatment works) of the system were investigated. Manning's equation was also compared qualitatively with the complete momentum equation for the hydraulic design of sewers.

Two mathematical models, SWMM and STORM, were employed. SWMM was used to design sewers and to analyze the characteristics of the quantity and quality of combined sewage and stormwater runoff for selected synthetic storms. STORM was used to analyze a continuous 12-year, real rainfall record to determine the frequency of overflow events, the mass of pollutants discharged in the overflows for the alternatives considered, and the duration and quantity of the overflows. The runoff and overflow characteristics were developed for one drainage district (about 265 hectares or 655 acres) in Elizabeth, New Jersey. A method was developed to use these characteristics to determine runoff and overflow characteristics from the remaining 24 drainage districts in the City so that a city-wide sewer system could be planned.

The study (a) evaluated the effects of differing rainfall patterns and intensities on pollutant concentrations, the effects of varying interceptor and treatment capacity on pollutants discharged, and the economics of peak flow equalizing for different levels of treatment; (b) quantified the long-term pollutional loads from combined sewage overflows; and (c) evaluated the effect on overflow control of varying the amounts of upstream and downstream storage. The report concluded that for Elizabeth, N.J., the most economical systems were those which included substantial amounts of upstream storage to minimize the size of trunk and interceptor sewers, pumping stations and treatment facilities.

While the study was made for the City of Elizabeth, New Jersey, the methodology should be generally applicable to other urban areas.

This report was submitted in fulfillment of Grant No. S-802971 by the City of Elizabeth, New Jersey under the partial sponsorship of the U.S. Environmental Protection Agency. Work was completed as of August 1976.

CONTENTS

Foreword....................iii
Preface....................iv
Abstract.................... v
List of Figures....................ix
List of Tables....................xii
Acknowledgments....................xv

Sections

I. Summary.................... 1
II. Specific Conclusions-City of Elizabeth.................... 5
    Pollution Characteristics - Combined Sewage.................... 5
    Pollution Control Potential - Conventional and Existing Combined System.................... 6
    Pollution Control - Advanced Combined Systems.................... 8
    The "Normalized Hydrograph" - a Planning Aid.................... 9
    Costs....................10
    Additional Hydraulic Studies....................13
III. Recommendations....................14
IV. Introduction....................16
V. Objectives and Scope of the Study....................19
VI. Description of Study Area....................21
VII. Suggested Methodology....................25
VIII. Design and Cost Comparison of Sewer Systems in District A....30
    Design Philosophy....................31
    Conventional Combined Sewer System....................35
        Design....................35
        Outflow Hydrographs and Pollutographs....................41
    Separate Storm Sewer System....................57
    Separate Sanitary Sewer System....................59
    Combined System with In-system Storage....................62
        Design with In-pipe Storage....................62
        Outflow Hydrographs and Pollutographs....................67
    Sewers with Satellite Storage....................67
        Design....................70
        Outfow Hydrographs and Pollutographs....................76
    Pollution Control Potential of Conventional Combined System....................83

Page

Cost Comparison of Alternative Collection Systems...... 83
Cost of Conventional Sewer Systems................ 85
Cost of Advanced Sewer Systems.................... 87

IX. Design and Cost Comparison of Alternative City-Wide Sewer Systems.. 89
Synthetic Storm Runoff................................ 89
Design of Intercepting Sewers......................... 96
Amount of Storage Required vs. Treatment Rate.......... 97
Cost Comparison of Alternative Sewer Systems...........100
X. Analysis of Overflow Quantity and Quality..................105
Effect of Storage and Interceptor Capacity.............106
Effect of Street Sweeping Practice.....................111
XI. Interceptors for Pollution Control.........................115
The "First Flush" Phenomena...........................115
XII. Alternative Pollution Control Programs.....................118
Conventional Combined System..........................118
Advanced Combined System..............................120
Selection of Design Storm for Separate and Combined Systems..124

XIII. Appendices...126

A. Alternative Model Evaluation.......................127
B. Modification of SWMM and STORM Programs............147

C. Quantity and Quality Considerations, SWMM vs STORM...150
D. Model Input Data...................................154
E. Calibration of STORM Runoff Coefficients...........197

XIV. References...202

XV. Glossary, Abbreviations and Symbols.........................206

FIGURES

Number Page

1 Study area, Elizabeth, New Jersey....22

2 Hyetograph and outflow hydrograph, District A (5-year intermediate pattern storm)....42

3 Mass rate SS pollutographs, 4 dry days (5-year intermediate pattern storm)....45

4 Mass rate BOD pollutographs, 4 dry days (5-year intermediate pattern storm)....45

5 Mass rate SS pollutographs, 30 dry days (5-year intermediate pattern storm)....47

6 Mass rate BOD pollutographs, 30 dry days (5-year intermediate pattern storm)....47

7 SS, BOD concentration pollutographs, 5-year intermediate pattern storm....49

8 Outflow hydrograph and pollutographs, 1-year intermediate pattern storm....50

9 Outflow hydrograph and pollutographs, 1.3-month intermediate pattern storm....51

10 Outflow hydrograph and pollutographs, 5-year advanced pattern storm....52

11 Combined sewage pollutographs for various storms....55

12 Stormwater runoff pollutographs for various storms....56

13 Sanitary sewer layout, subareas I and II....60

14 Sanitary sewer layout, subareas III, IV and V....61

15 Incremental cost of pipe storage....63

FIGURES (continued)

Number		Page
16	Weir-orifice regulator	64
17	Hydrographs modified by in-pipe storage	66
18	Outflow hydrographs with in-pipe storage (5-year intermediate pattern storm)	68
19	Wet-weather BOD variation, Trenton, New Jersey	69
20	Outflow hydrographs with satellite storage at various locations (5-year intermediate pattern storm)	71
21	Schematic diagram of satellite storage locations, District A	73
22	Outflow hydrographs with satellite storage (5-year intermediate pattern storm)	75
23	Outflow pollutographs with satellite storage (5-year intermediate pattern storm)	77
24	Outflow hydrographs and pollutographs with satellite storage (1-year intermediate pattern storm)	79
25	Outflow hydrographs and pollutographs with satellite storage (1.3-month intermediate pattern storm)	80
26	Outflow hydrographs and pollutographs with satellite storage (1-year advanced pattern storm)	81
27	Normalized hydrographs	93
28	Outflow hydrographs, Districts A, E and H	95
29	Total cost comparison-city sewer system, in-pipe vs. satellite storage	103
30	Correlation of annual SS overflow with annual precipitation	107
31	Annual number of overflow events for various storage and interceptor capacities	109
32	Annual SS overflow for various storage and interceptor capacities	110
33	Effect of street cleaning interval on SS street washout	113

FIGURES (Continued)

Number Page

34 Pollution control costs - conventional system...............121

35 Pollution control costs - advanced system...................123

A-1 Definition sketch for equations of motion...................128

A-2 Uniform flow rating curve and loop rating curve.............132

A-3 Surcharged simple pipe system...............................135

C-1 Street SS accumulation, SWMM vs. STORM......................153

D-1 Frequency distribution of precipitation amounts.............156

D-2 Frequency distribution of precipitation duration............157

D-3 Frequency distribution of antecedent dry hours..............158

D-4 Rainfall intensity-duration-frequency curves................160

D-5 5-year design storm hyetographs and infiltration curve......161

D-6 Frequency of hourly rainfalls...............................163

D-7 Subareas of drainage District A.............................166

D-8 Combined sewer system layout, subarea I....................167

D-9 Combined sewer system layout, subarea II...................168

D-10 Combined sewer system layout, subarea III..................169

D-11 Combined sewer system layout, subarea V....................169

D-12 Combined sewer system layout, subarea IV...................170

D-13 Separate storm sewer system layout.........................171

D-14 Definition of drainage districts and interceptor system in Elizabeth...186

D-15 Unit cost of sewers..193

D-16 Cost of treatment vs. % of BOD removal.....................196

E-1 STORM runoff coefficient C_1 vs. rainfall..................200

TABLES

Number	Page

1 Lateral Sewer Design Diameters (inches) 37

2 Trunk Sewer Design Diameters (feet) 40

3 Comparison of Design Diameters and Peak Hydrographs 43

4 Solids Deposit (lbs) in Sewers with Four Antecedent Dry Days 46

5 Summary of Runoff Quantity and Quality Characteristics for Various Storms 54

6 Comparison of Pipe Diameter, Slope, and Excavation Depth, Separate Storm versus Combined Systems 58

7 Peak Flow Comparison for Separate Storm and Combined Sewer Design 59

8 Comparison of Trunk Sewer Sizes (feet) for Storage Basin at Various Locations 72

9 Summary of Runoff Quantity and Quality for Combined System with Satellite Storage 82

10 Storage Volume in Sewer Pipes and Storage Basins, District A 84

11 Cost Comparison of Conventional Combined and Separate Sewer system ($$10^6$) of District A 86

12 Cost Summary of Combined System with In-system Storage 88

13 Summary of Normalized Hydrograph Development from District A Data 91

14 Adjustment of Peaking Time with Respect to that of District A 92

15 Development of Outflow Hydrographs 94

TABLES (continued)

Number Page

16 Intercepting Sewer Pipe Element Data.....96

17 Summary of Intercepting Sewer Diameters (feet) and Cost ($$10^6$).....98

18 Off-line Storage Required (10^6 gallons) and Cost for Equal Weight of SS Discharged to Receiving Waters.....99

19 Total Cost of City-Wide Sewer Systems.....101

20 Effect of Storage and Flow Interception on District A Sewer System Discharges.....108

21 Pollutant Overflow as Affected by Street Sweeping Interval and Efficiency.....112

22 Interceptor Capacity Required to Contain First Flush from District A.....116

23 Comparison of Alternative Combined Sewer Systems.....119

A-1 Range of Typical Values for Each Term in Momentum Equation.....130

A-2 Peak Flow Comparison between Rational Method and SWMM.....144

C-1 Pollutant Default Values for SWMM and STORM.....151

D-1 Annual Precipitation and Runoff Events.....155

D-2 Hourly Rainfall versus Return Interval.....164

D-3 Storm Characteristics.....164

D-4 Area of Land Use Distribution in Each Subarea (acres).....165

D-5 Total Sewer Length Comparison.....172

D-6 Subcatchment Data for SWMM.....173

D-7 Pipe Element Data for Combined Sewer System.....177

D-8 Subcatchment Water Quality Input Data for SWMM.....179

D-9 Subarea Curb Length (feet) versus Land Use.....182

TABLES

Number Page

D-10 Summary of Drainage District A Land Use Data....182

D-11 Monthly Evaporation Rate....184

D-12 Summary of Land Use in The City of Elizabeth....187

D-13 Population....188

D-14 Average Daily Pollutant Concentration of Domestic Wastewater....189

D-15 Diurnal Variation of Domestic Wastewater Quantity and Quality for District A....190

D-16 SWMM Input Data for Domestic Wastewater Computation, District A....191

D-17 Unit Cost of Sewers....194

E-1 Sensitivity of Hyetographs and Integration Interval on Runoff Volume....198

E-2 Calibration of Runoff Coefficient C_1....199

ACKNOWLEDGEMENTS

The support of the project by the City of Elizabeth and the Office of Research and Development, U.S. Environmental Protection Agency is acknowledged and appreciated.

The assistance and cooperation of Messrs. Richard Field and Anthony N. Tafuri, Chief and Grant Project Officer, respectively, Storm and Combined Sewer Section, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency, has been invaluable.

This project was directed by Mr. Herbert L. Kaufman, Partner-In-Charge. Messrs. John H. Scarino, Principal Associate and Director of Engineering Management, and William Wheeler, Associate, provided valuable criticism and review. Dr. Fu-hsiung Lai served as Project Engineer and Mr. Gerald G. Gardner as Engineer. Dr. Lai developed original and innovative analyses, which resulted in significant findings as to practical methods for reducing pollution from combined sewage overflows.

Dr. Brendan M. Harley, Dr. Guillermo J. Vicens and Mr. Richard L. Laramie of Resource Analysis, Inc., Cambridge, Massachusetts, provided computer program changes to SWMM and STORM and prepared drafts for parts of Appendix A in late 1974. Dr. Harley also participated in the preparation of the "Design Philosophy" described in Section VIII.

Dr. Wayne C. Huber of the University of Florida and Mr. Harry C. Torno of the U.S. Environmental Protection Agency reviewed the draft of this report. Their comments and constructive criticism are appreciated. Dr. Huber also freely contributed his expertise in many discussions during the preparation of the report for which we express our sincere thanks.

Finally, the authors are grateful to Mr. M.B. McPherson, Director of ASCE Urban Water Resources Research Program, and Mr. Chi-Yuan Fan, Storm and Combined Sewer Section, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency, who also contributed constructive comment.

SECTION I

SUMMARY

Pollution resulting from the discharge of untreated wastes from combined sewers is a national problem which requires correction if the objective of clean waters is to be achieved. Such discharges are now experienced from systems serving a population of 54 million located in some 1,300 municipalities. They occur, on average, about every 5 to 10 days, depending on location, and can reduce the efficiency of pollutant removal from about 90 percent to 15 to 25 percent.

Using computer technology, tools to investigate and analyze the various elements required for planning cost-effective solutions for combined sewage overflow pollution abatement have been developed. This study modifies and improves existing tools and develops new tools. It organizes these tools into a systematic methodology that can have broad application.

This study developed appropriate criteria for sewerage improvements. The criteria are to be used to:

- identify alternative corrective programs for abating pollution from combined sewage overflows, and
- select the most cost-effective solution.

STUDY PROCEDURE

Initially, alternatives for combined and separate sewage collection systems to serve a 1.6-square-kilometer (1.0-square-mile) area in Elizabeth were analyzed. The systems were designed to convey, without flooding, flows resulting from a two-minute interval, synthetic storm hyetograph which matched the intensity-duration curve applicable to the area at each time interval for a 5-year return frequency rainfall. The characteristics of the hydrographs and pollutographs for the alternative collection systems were determined and applied to the remaining 11.0-square-kilometer (6.9-square-mile) urban area of the City. The alternatives, including interceptor and treatment/storage systems, which would provide essentially complete capture and treatment of pollutants for each of the collection system alternatives, were then screened by comparing costs of the total systems. The reduced cost resulting from reducing the amount of pollutant captured for treatment were then estimated and an evaluation made of the pollutional impact of the resulting discharges.

The 5-year return interval, synthetic storm hyetograph developed does not represent the 5-year return frequency runoff. However, since this study emphasized pollution abatement, and, since the calculated amounts of mass pollutants in the runoff from a similarly synthesized 1-year return interval storm hyetograph are not significantly different (TABLE 9), the amount of pollutants determined may be expected to represent the quantity to be expected from a 5-year return frequency runoff volume computed by other methods. The function served by the selected 5-year synthetic storm hyetograph was to permit design of alternative facilities that were tested for effectiveness in pollution abatement with a 12-year continuous real rainfall record.

For the City of Elizabeth, the quantity of pollutants in urban runoff and combined sewage which can be intercepted for treatment with alternative facilities were relatively quantified. The alternatives considered included:

1. Separate sewer systems;
2. Combined sewer systems designed to convey urban runoff from a synthetic, 5-year return frequency rainfall (Figure D-5);
3. Combined sewer systems with flow routing and storage in the trunk sewer;
4. Combined sewer systems with enlarged lateral sewers to permit storing various amounts of runoff from an intermediate pattern storm (peak rainfall in the middle half of the storm duration);
5. Combined sewer systems, similar to 4, except that storage basins were provided along the trunk sewer; and
6. Interceptor sewers of varying capacity together with storage and treatment facilities.

Evaluation of 32 alternatives for collection system modifications and of 27 alternatives for total sewer system modifications was made, assuming all new construction, to determine relative cost-effectiveness. The interdependent relations between flood alleviation and pollution control in a combined sewer system were described.

METHODOLOGY

A suggested flexible methodology has been developed which can be applied for analysis and screening of many alternatives to determine the cost-effective solution for abatement of combined sewer overflows. It modifies to some extent the procedures used in this study. The methodology did not develop a "design storm" for pollution control. It used a long-term, continuous rainfall record and calculated runoff and quality characteristics of real rainfalls to determine the relative effectiveness of alternative facilities.

The EPA Storm Water Management Model (SWMM) was used to:

1. design the many alternative systems for a common synthetic rainfall event (Figure D-5), and
2. quantify relatively the quality and quantity of combined sewage for a broad range of specific, lesser, synthetic rainfall events with varying patterns.

The Corps of Engineers' Storage, Treatment, Overflow and Runoff Model (STORM) was used to compare the performance of the various systems quantitatively with respect to pollution abatement for a 12-year continuous period of real rainfall events. It determined:

1. the number of annual combined sewer overflow events;
2. the mass loading (weight) of pollutants discharged in the overflows for the different facilities considered; and
3. the duration and quantity of the overflows.

The study evaluated the effects of:

1. random rainfall patterns and intensities;
2. the long-term and shock pollutional loads from combined sewage overflows;
3. varying amounts, modes of operation, and locations of storage;
4. varying interceptor and treatment capacity;
5. peak flow equalization; and
6. rainfall patterns on pollutant concentration.

The SWMM RUNOFF Block was modified to include design capability in sizing sewers. STORM was modified to extend its use to combined sewage.

SIGNIFICANT FINDINGS

1. Capture of the low-volume, high-concentration first flush from combined systems is essential for pollution abatement.
2. Annual discharges of moderate rainfalls contain more pollutant with higher concentrations than do discharges from severe storms. The combined sewage discharges of moderate rainfalls should be controlled and treated for protection of the environment.
3. Storage of combined sewage should reduce pollutant concentration as a result of mixing the highly polluted first flush with later,

less polluted flows. Storage is effective in abating pollution.

4. If storage cannot be provided, diversion of combined sewage flows of 17 or more times the peak dry-weather flow for treatment should result in significant pollution abatement benefits.

5. A combined system can be designed to discharge less pollutant than a conventional separate system in which the storm sewers discharge all urban runoff directly to water courses.

6. Providing additional storage in a combined sewer system should result in the least-cost system for pollution control. This storage can be provided by enlarging the size of the upstream sewer reaches if site restrictions result in costly satellite storage basins.

The ability of SWMM and STORM to simulate the pollutant components reliably in urban runoff and combined sewage is questionable. This also applies to all known presently available models. However, the "first flush" phenomenon has been observed, as well as the substantial reduction in pollutant concentrations after the "first flush". Hence, the relative quantification of pollutants, through SWMM and STORM, offers a practical, available measurement by which alternatives for combined sewage overflow abatement facilities can be developed and evaluated.

SECTION II

SPECIFIC CONCLUSIONS - CITY OF ELIZABETH

POLLUTION CHARACTERISTICS - COMBINED SEWAGE

1. The magnitude of pollution resulting from combined sewage overflows in Elizabeth is indicated by:

    (a) the probability of about 66 overflows of raw, combined sewage annually, or about one every 5 or 6 days;

    (b) an average of about 60 percent of the raw sewage entering the sewer system during the overflow periods being discharged untreated; and

    (c) the concentration of pollutants in the initial discharges possibly being an order of magnitude greater than in normal dry-weather flow.

2. Combined sewage generated by an "intermediate" (typical) pattern storm (peak rainfall in the middle half of the storm duration) exhibits two peak flushes of mass pollutants. The first peak flush includes the washout of solids deposited in combined sewers from sanitary wastes during dry days. The second peak flush includes the washout of street pollutants. The first peak is of low-flow volume but high-pollutant concentration. The second peak is of high-flow volume but low-pollutant concentration.

    Combined sewage generated by an "advanced" pattern storm (peak rainfall in the first quarter of the storm duration) exhibits only a single peak flush. When compared to the peaks of an intermediate pattern storm, this peak has less flow volume but a much higher pollutant concentration than the second peak and a lower concentration than the first peak.

    With either storm pattern, however, the highest concentration of pollutants occurs early in the storm.

3. The mass washout of combined sewage solids from an individual major storm (5-year rainfall return frequency) is greater than from individual frequent rainfalls (1.3 and 0.35-month rainfall return frequencies). However, the total pollution resulting

from frequent rainfalls is greater. Rainfalls with an individual cumulative depth of less than 25.4 mm (1.0 inch) produce about 93 percent of the total annual precipitation and about 73 percent of the total pollutant washout for the year. A storm of 0.35-month (about 10 days) return frequency, which has a peak runoff rate about ten times the average dry-weather flow, should cleanse the sewer of most solids deposits. Hence, capturing runoff from frequent storms is essential for pollution control.

4. A 5-year return frequency rainfall may be expected to remove about 96 percent of the total suspended solids accumulated on streets; a 1-year rainfall, 87 percent; and a 1.3-month rainfall, 35 percent. However, except when preceded by a protracted dry period, street washout should contain substantially lower pollutant concentrations than the sewer solids washout since it usually occurs at a time when high flow rates provide dilution.

5. With the synthetic 5-year storm hyetograph used, treatment of essentially all pollutants in the combined sewage and street washout would require capture for treatment of the first 90 minutes of flow from the storm. Such facilities may be justified only where the receiving water requires an extremely high quality discharge to preserve the environment.

6. In a combined sewer system, the lower the flow velocity in dry weather, the greater would be the amount of solids deposit. Maintaining or creating flushing velocities in combined systems during dry weather should provide a significant reduction in the pollutant concentration in the initial storm flush.

7. Both SWMM and STORM provide an option with internally specified default values (see TABLE C-1) for computation of suspended solids (SS), biochemical oxygen demand (BOD), and coliforms from the mass of dust and dirt in the street washouts. Use of the SWMM default values results in SS and BOD mass discharges about ten and five times, respectively, that computed using STORM default values.

POLLUTION CONTROL POTENTIAL - CONVENTIONAL AND EXISTING COMBINED SYSTEMS

1. Flow routing devices can be used to reduce the number of annual combined sewage overflow events. When installed in a conventional combined sewer system, designed using SWMM for a synthetic intermediate pattern storm, and having interceptor capacity equal to the peak dry-weather flow, reductions for Elizabeth proved to be as follows:

Design Storm Return Frequency	Reduction of Overflow Events	Mass Reduction of Overflow Pollutants
10 years	80%	75%
5 years	65%	55%
2 years	small	small

The ineffectiveness of the system designed with a 2-year return interval storm in abating pollution results from the reduction in volume of about 40 percent for storage of flows during a 1.3-month storm. Analysis of real rainfall events, based on a continuous 12-year record, indicates the available storage volume would be exceeded frequently, substantially increasing the number of overflow events and the mass of pollutants discharged.

The cost of sewer facilities to prevent flooding from a 10-year return frequency rainfall is about 25 percent greater than from a 5-year return frequency rainfall. Since less costly means are available in the design of new systems to reduce pollution from combined sewage overflows and urban runoff, the use of a 10-year return interval storm for pollution control can not be justified.

2. Increasing interceptor sewer size can reduce the number of overflows and pollutants discharged in combined sewage overflows. Based on analysis using actual rainfall events over a 12-year continuous period, the calculated relative amounts of SS and BOD discharged for various interceptor capacities were determined. The analysis assumed no storage upstream of the interceptor. The results for a drainage area of about 1.6 square kilometers (1.0 square mile) are summarized as follows:

Interceptor Capacity (Times Peak Dry-Weather Flow)	Average Annual Overflows			
	Number of Events Longer than 1 Hour	Volume (%)	Relative Amounts of Pollutants SS (%)	BOD (%)
1.0*	65.8	100.	100.	100.
3.8	40.3	50.40	41.70	29.30
6.2	28.5	32.50	26.30	15.00
8.2	18.5	24.20	18.10	10.00

(continued)

* Conventional design capacity based on Harmon's Ratio.

Interceptor Capacity (Times Peak Dry-Weather Flow)	Number of Events Longer than 1 Hour	Volume (%)	SS (%)	BOD (%)
16.5	6.6	8.90	5.20	2.50
22.3	3.8	4.30	2.30	1.10
23.1	3.5	4.00	1.90	0.90
26.8	2.3	3.30	1.20	0.60
59.2	0.2	0.04	0.04	0.04
151.7	0.0	0.00	0.00	0.00

The mass of pollutants discharged reduced at a faster rate than the volume due to greater capture of the initial flush.

3. Where the capacity of an existing combined system is such as to result in frequent combined sewage flooding, but where the system is adequate to convey dry-weather flows, and adequate storage facilities cannot be provided economically upstream, it is cost-effective by a large margin to divert urban runoff in excess of the combined system capacity to a separate stormwater system for the same level of pollution abatement.

4. Based on data developed for Elizabeth, use of the Rational Method to determine peak runoff rates results in values 30 to 60 percent less than those obtained using SWMM for a synthetic 5-year return frequency storm (Figure D-5). In a separate study by the U.S. Army Corps of Engineers at Louisville, Kentucky in 1949 (33), peak runoff rates obtained with the Rational Method were 20 to 43 percent less than those obtained with the Unit Hydrograph Method. Sewers sized in accordance with such values would not permit effective use of flow control devices for pollution control because of the limited storage volume available.

POLLUTION CONTROL - ADVANCED COMBINED SYSTEMS

1. Storage of wet-weather flows in the collection system, either in the pipe (in-pipe) or in storage basins (satellite), is an effective pollution control method. By mixing the highly polluted initial flushes with the later, less polluted runoff, the concentration of pollutants discharged in the untreated wastes is reduced. By reducing the peak flow rate, more of the waste can be diverted to the interceptor and subsequent treatment. The following presents the SS diverted to treatment for the various synthetic storms selected with varying amounts of storage (also see TABLE 9):

Rainfall Return Frequency	SS Diverted in Percent			
	0*	9.4*	17.0*	41.3*
5 years	0	34	37	51
1 year	0	37	46	78
1.3 month	0	55	56	91

* Storage in percent of 5-year synthetic storm runoff volume. The pollution control benefits increase with the amount of storage provided and with a shorter return interval storm.

2. In a combined system, upstream storage reduces the size of trunk, interceptor and treatment facilities. For a 1.6-square-kilometer (1.0-square-mile) area in Elizabeth, about 99 percent of the pollutants that would overflow from a conventional combined system could be captured for treatment with storage equal to 9.4 percent of the 5-year synthetic storm runoff and an interceptor capacity of 17 times peak dry-weather flow. If the storage is increased to 41.3 percent, the interceptor capacity could be reduced to about four times peak dry-weather flow for the same amount of capture. With 41.3 percent storage and interceptor capacity equal to the peak dry-weather flow, (the conventional capacity), about 95 percent of the pollutants should be captured for treatment. Storage for pollution control provides flood protection as a side benefit.

3. The manner of flow routing through storage influences both costs and the degree of pollution control achieved. If upstream storage is bypassed until the downstream trunk sewer capacity is exceeded, interceptor and downstream storage and treatment capacity must be much larger for a desired pollution abatement goal as compared to the facilities required if all flow is routed through upstream storage. For a given facility, bypassing upstream storage basins until the trunk sewer capacity is reached would result in lowest operating costs, but routing all flow through upstream storage basins would result in greatest pollution control. The preferred method of operation might consider bypassing upstream storage until interceptor capacity is exceeded.

THE "NORMALIZED HYDROGRAPH" - A PLANNING AID

1. For a given rainfall pattern, the primary factors defining the outflow hydrograph of a combined drainage system serving an urban environment, are area and percent imperviousness. The area influences peak flow reduction and time of occurrence. Percent imperviousness affects flow volume. Within reasonable ranges of area and percent imperviousness, the shape of the hydrographs is sufficiently similar to permit development of a family of

"normalized hydrographs" whose ordinates are defined by a percentage of the peak runoff rate and whose area is measured in seconds. The volume of runoff of a storm event may be determined by a simple computation (as in STORM). The runoff at each ordinate then is determined so that this volume is obtained. These ordinates provide an adequate representation of the hydrograph from the given area for flow routing through an interceptor. This concept is an aid in reducing the amount of segmentation required and the costs for planning.

COSTS

1. The amount of in-pipe storage which may be provided is limited by cost since:

    (a) The incremental unit cost of in-pipe storage volume increases with sewer size (Figure 15),

    (b) storage resulting from a large increase in sewer diameter should be more costly per unit of volume than for a relatively small increase,

    (c) pipe storage in the smaller lateral sewers provides not only lower costs for in-pipe storage but also permits reduction in the size of the more costly trunk sewers, and

    (d) satellite (off-line) storage, which is located so as to reduce trunk sewer sizes, would probably be a cost-effective alternative since its cost per unit of volume may be as little as 30 percent of in-pipe storage.

2. Storage to reduce costs should result in:

    (a) reduction of the peak flow of the outflow hydrograph to the extent practical and economical,

    (b) the multiple peaks of the hydrograph being relatively equal, and

    (c) reduction of the concentration of pollutants in the initial peak by dilution.

3. Comparative costs of the alternative collection systems considered are as follows (TABLE 11, TABLE 12):

    (a) a conventional combined system should cost about the same as separate systems when surcharge of the separate drainage system is permitted,

    (b) the combined system permitting significant overland drainage flow is the least cost conventional system,

(c) separate systems in which surcharge of the storm drainage system is not permitted are most costly,

(d) surcharge of a separate storm drainage system reduces its costs by about 20 percent,

(e) in the case of the City of Elizabeth storing about eight percent of the 5-year synthetic storm runoff volume in the upstream lateral system results in the least cost for in-pipe storage systems since the increased cost of the lateral sewers is more than compensated by the reduced cost of trunk sewers. However, the increased cost resulting from doubling or tripling the amount of storage in the laterals is relatively small,

(f) if satellite storage basins can be built at low cost, about \$0.08 per liter (\$0.3 per gallon), the least-cost collection system results from storing about 17 percent of the 5-year synthetic storm runoff volume, and

(g) if satellite storage is costly, about \$0.26 per liter (\$1.0 per gallon), the cost of the collection system increases with the increase in storage volume.

4. Comparative costs of the alternative systems considered for control of urban runoff and combined sewage overflows from the entire City, assuming essentially complete capture (92% using the 5 year synthetic hyetograph) of pollutants in both sewage and urban runoff for treatment, are:(TABLE 19 and Figure 29)

   (a) Where the cost of off-line storage is high, \$0.26 per liter (\$1.0 per gallon), and in-pipe storage cannot be provided, separate sewers would be more economical than combined sewers. However, where the storage cost is low, \$0.08 per liter (\$0.3 per gallon), the combined system is more economical.

   (b) The amount of in-pipe storage that may be optimally provided is about 30 percent of the runoff developed using the 5-year synthetic storm hyetograph with the assumed range in cost of off-line (downstream) storage.

   (c) The conventional separate system would be the most costly choice if storage (off-line or satellite) can be built at a cost of \$0.08 or less per liter (\$0.30 per gallon).

   (d) At a satellite storage cost of \$0.26 per liter (\$1.0 per gallon), the conventional separate system would be more economical than any alternative advanced combined system.

   (e) In all instances, the advanced combined system offers advantage over the conventional system.

5. The least-cost system for essentially complete control of pollutants from urban runoff and combined sewage is expensive, being estimated at more than $114,000 per hectare ($29.5 million per square mile) (ENR Construction Cost Index equals 1800).

6. The criteria of removing essentially all pollutants from urban runoff and combined sewage overflows is economically impractical. Capturing about 97.5 percent of the wet-weather BOD for treatment, rather than substantially 100 percent, would reduce costs of the overall system by about 25 percent. Hence, alternative criteria were explored which would result in acceptable pollution control at a more reasonable cost. These are tabulated below:

Description	Interception Rate % of Peak DWF	Pollutant* Capture %	Cost $/Hectare
A	380	58	79,000
A	2310	98	96,000
B	1380	98+	90,000
C	220	98	80,000
A	100	0	75,500

A. Conventional combined system.

B. Conventional combined system with flow routing to utilize storage capacity available in sewers during normal rainfalls.

C. Advanced combined system with about 40% in-system storage.

* With respect to SS overflow from a conventional combined system with intercepting rate equal to peak dry-weather flow.

A conventional combined system, without provision for pollution control, would cost about $75,500 per hectare ($19.6 million per square mile). Upgrading this system by increasing interceptor capacity from 1.0 to 3.8 times peak dry-weather flow would increase costs by about five percent but provide about 58 percent pollutant capture. Increasing interceptor capacity, only, to obtain 98 percent pollutant capture would increase base costs by about 27 percent. Using a conventional system with flow routing and increased interceptor capacity to obtain a pollutant capture of 98 percent would increase the base cost by about 20 percent. If in-system storage equal to about 40 percent of the 5-year synthetic storm runoff can be provided, 98 percent of the annual pollutants in the combined wastes should be captured with an increase of base cost of about seven percent.

The cost of providing pollution control by in-system storage and flow control devices for urban runoff and combined sewage is quite small when considering the overall investment in sewerage systems.

7. The optimum cost relation between off-line storage and treatment capacity depends on the degree of treatment required. If simple processes such as microstraining will suffice, the cost of additional storage would about balance the cost of reduced treatment facilities so that the total cost is about the same within any reasonable range (TABLE 19). Should more sophisticated treatment be required, larger storage and smaller treatment facilities would afford greatest economy.

ADDITIONAL HYDRAULIC STUDIES

1. Overland flow can be adequately modeled by retaining only the bottom slope and friction slope terms in the complete momentum equation (Kinematic Wave Model).

2. Simulation of pipes in urban system may require that the gravitational and convective acceleration terms be retained, as done in the SWMM TRANSPORT Block over individual pipe segments. These terms will introduce dynamic effects which exist in urban sewer routing.

3. In cases where backwater effect (say, near a control structure) or surcharging of pipes is present, it is inappropriate to use the assumption that the pipe slope and the friction slope are equal. Manning's equation can only be used in these cases if the appropriate friction slope is computed. But in these cases, the friction slope depends on flow conditions and is not known in advance. In the absence of such effects, use of Manning's equation and assuming friction slope equals pipe slope, appears to be a good approximation in sewer design.

4. No detailed comparison of cost reduction versus number of terms modeled for the momentum equation has been published. Most evaluations and comparisons have focused on overall differences between models and not on a comparison of the details of the solutions to the basic equations. However, models which include more than the bottom and friction slope terms would increase the cost of computer use. These "full equation" models need be used only when extreme accuracy can be justified.

   However, other factors such as inaccurate rainfall data, inaccurate modeling of control structures, and the modifications used to model a prototype system with a limited set of model elements, may produce errors more significant than those introduced by using a simple model such as the Kinematic Wave Model which neglects the gravitational and convective acceleration terms in momentum equation.

SECTION III

RECOMMENDATIONS

1. The Rational Method should be used with extreme caution. It should not be used for systems in which controlled routing or storage for pollution control is desired.

2. In urban areas with relatively high imperviousness, combined sewers should be considered for new development.

3. New combined sewers should be designed to maintain flushing velocity for dry-weather flows.

4. Urban runoff and combined sewage overflow pollution control should be predicated on containing pollution from the less intense, frequent storms rather than from intense rare storms.

5. The storage contained in trunk sewers of a combined sewer system designed with SWMM for a 5-year or greater return frequency storm hyetograph should be exploited by installing suitable regulators for pollution control of runoff from less intense, frequent storms. Development of simple inexpensive regulators for utilization of in-pipe storage is needed.

6. Upstream storage in the collection system (either in-pipe or satellite) should be required in new combined systems and, where practical, for the improvement of existing systems.

7. More reliable quantification of runoff in urban areas would be desirable to determine absolute rather than relative quantities of pollutants discharged in urban runoff and combined sewage overflows. Further investigations might include street washout mechanisms, pollutant constituents in dust and dirt, and the contribution of solids deposits in combined sewers to BOD loading.

8. Based on computed data, control of first flush pollutants appears to be a better criterion for Elizabeth than control of the total mass of pollutants.

9. To achieve pollution control benefits from controlled routing in Elizabeth, design based on a 5-year return interval synthetic storm

hyetograph using SWMM can be justified. Use of a 2-year return interval storm largely forfeits the benefits of flow routing, and the additional storage obtained from using a 10-year return interval storm may be more economically achieved by satellite basins.

SECTION IV

INTRODUCTION

Concern with storm runoff primarily has considered street and basement flooding. Sewers were installed to correct such problems. With recognition of the pollutants contained in storm runoff, reduction of the amounts of pollutants reaching natural water bodies has become a significant factor. The pollution load placed on a stream or other receiving waters by combined sewer overflows is substantial. It has been estimated in a study conducted at Northhampton, England, that the total pounds of BOD contributed yearly to receiving waters by storm generated overflows may be equal to about 15 percent of the BOD in the dry-weather flows (1). A U.S. Environmental Protection Agency (EPA) funded study in Durham, North Carolina, resulted in similar findings (2).

The earliest sewers were built to convey stormwater runoff. They were converted to combined sewers in later years as waterborne wastes were discharged to them. Subsequently, the use of combined sewers became widespread. A nationwide survey by the American Public Works Association (APWA) indicated that drainage areas totalling over 1,214,100 hectares (3,000,000 acres) in more than 1,300 municipalities and a population of 54 million are served by some 875,000 kilometers (550,000 miles) of combined sewers. Of the 641 jurisdictions surveyed, 493 reported about 14,200 combined sewer overflow points. Three hundred and forty reported infiltration problems during wet weather and 96 indicated combined sewer overflows during dry weather (3). The cost of sewer separation in the U.S. has been estimated at about $96 billion in 1974 (4). The merit of sewer separation has been argued with respect to effectiveness in storm runoff pollution abatement (4).

Elimination of all pollution from wet-weather flow could be prohibitively expensive. There is, however, an optimal level of expenditure for pollution control that should be justified by environmental benefits. To enhance the urban environment and to conserve water and other resources, the design of either combined or separate sewer systems should:

1. provide sufficient relief of street and basement flooding from urban runoff or combined sewage,

2. limit pollution from municipal wastes and urban runoff that may enter natural waterways to an acceptable level that will not adversely affect the environment, and

3. be economically feasible in context with society's needs.

Alternatives for the alleviation of street flooding include:

1. gutter inlets and collection sewers of adequate capacity to collect and convey the defined peak urban runoff,
2. storage basins for the containment of urban runoff to reduce peak flow entering or flowing in sewers,
3. a combination of collection sewer capacity and in-pipe storage, and
4. source control, including porous pavement to reduce runoff volumes, and roof detention or ponding over designated areas to reduce peak flows.

Alternatives for pollution control in the natural waterways from wet-weather flow include:

1. increased interceptor and treatment capacity to convey and treat wet-weather flows before discharge,
2. off-line storage for equalization of the treatment rate and reduction of capital investment in interceptors and treatment plants,
3. in-pipe storage for containment of pollutants in runoff and for modulating the runoff hydrograph of flow to the interceptor system, and
4. source control, including sewer flushing to reduce solids deposit in sewers, street sweeping to reduce the pollutant washout and/or porous pavement to reduce the runoff quantity.

The alternatives for alleviation of urban flooding and for urban runoff and combined sewage pollution abatement are interdependent. The upstream sewer facilities planned for flood control determine the character of the flow hydrographs and pollutographs reaching the downstream interceptor, storage and treatment facilities and impacting on the receiving water. Upstream storage, source control or other quantity control measures have potential benefits in both pollution and flood control. A plan for a community-wide sewer collection system should include optimum exploitation of those benefits.

The complex nature of the urban rainfall/runoff and the pollutant accumulation/washout/transport processes, and evaluation of the many management alternatives have made use of computer models advantageous. Traditionally, storm sewers are designed using the Rational Method. It provides an estimated peak flow but not the runoff hydrograph and pollutograph needed for the evaluation of runoff pollutants and the design of facilities for their control and treatment. Several more recent design methods for drainage systems have been proposed (5,6,7). These methods employ momentum equations for computing unsteady storm runoff in overland catchments and in sewers and utilize the micro (fine discretization) approach in the application of hydrologic and hydraulic theory.

This study was undertaken to analyze the effects of alternatives available for the design of sewer systems considering both pollution and flood control. The alternatives were analyzed for the urban area of Elizabeth, New Jersey, but the principles and general conclusions should be applicable to other urban areas.

SECTION V

OBJECTIVES AND SCOPE OF THE STUDY

This study compares:

1. sewer system design flows using conventional and advance hydrologic and hydraulic methodology, and

2. the cost-effectiveness of

   (a) a conventional separate storm and sanitary system,

   (b) a conventional combined system, and

   (c) advanced combined systems with varying amounts of in-pipe and/or satellite storage and controlled flow routing.

The cost-effectiveness comparison was initially based on achieving a very high standard of pollution abatement. As the study progressed, it became apparent that to provide an effective basis for evaluation of alternatives for pollution abatement, the scope required extension to evaluate facilities for less complete, but acceptable, pollutant capture for treatment.

The components of a sewer system include collection sewers, storage (both in-pipe and satellite) facilities in the collection systems, interceptors, and downstream off-line storage and treatment facilities. New systems were assumed to provide a uniform basis of comparison between the alternatives and permit application of the findings to other cities. However, the study also includes the case (applicable to Elizabeth and other cities) in which the existing system can be used as a sanitary system and a new storm system provided for pollution abatement and alleviation of combined sewage flooding.

The scope of the study includes:

1. comparison of sewer design equations,

2. review and selection of mathematical models for evaluation of alternatives,

3. development of synthetic storm hyetographs, hydrographs and pollutographs,

4. alternative sewer designs at a master planning level for Drainage District A of the City of Elizabeth, with an area of 265 hectares (655 acres),

5. economic evaluation of the alternative designs for Drainage District A,

6. development of quantity and quality parameters for application to the entire City,

7. design of interceptors and downstream facilities for pollution abatement,

8. cost-effective comparison of the alternative systems for the entire City,

9. determination of the facilities required to provide an acceptable level of pollution abatement, and

10. the development of a methodology for planning facilities for pollution abatement.

SECTION VI

DESCRIPTION OF STUDY AREA

The City of Elizabeth, New Jersey (Figure 1) encompasses a total urban area of about 1,780 hectares (4,400 acres) and has been divided into 25 drainage districts for planning sewerage improvements. The existing sewers are largely combined. Approximately 90 percent, by length, of the sewers are from 50 to 100 years old, and the expected deterioration in their condition has occurred. This is evidenced by occasional sewer collapses, joint deterioration, and physical inspection when sewers are exposed in excavations. In 1973, the base sewage flow in the City was estimated as follows:

Flow Category	Amount (mgd)
Residential	6.5
Commercial	1.1
Industrial	3.4
Municipal	0.6
Total	11.6

There are 37 points in the system at which combined sewage overflows to natural water courses during wet weather. In 1973, the principal overflows on the Westerly Interceptor discharged combined sewage during 61 storm events while the principal overflows on the Easterly Interceptor discharged sewage during 44 storm events. Discharges from the Westerly Interceptor entered the Elizabeth River while those from the Easterly Interceptor entered the Peripheral Ditch around Newark Airport, the Great Ditch, Newark Bay, the Arthur Kill and the Elizabeth River. Overflows during dry weather were experienced at Westfield Avenue, the upstream terminus of the Westerly Interceptor. These overflows may extend for about 18 hours per day and may be the result of a blockage in the Westerly Interceptor or of very high infiltration in its tributary area.

Average infiltration in 1973 was estimated as 0.28 m^3/sec (6.4 mgd), or about 8.7 million m^3 (2.3 billion gallons) per year. Infiltration equals about 55 percent of the average dry-weather flow and indicates the deterioration of the system with age. Excluding dry-weather discharge of wastes at Westfield Avenue, the total amount of infiltration from Elizabeth treated at the Joint Meeting Plant was about 7.6 million m^3 (2.0 billion gallons) in 1973.

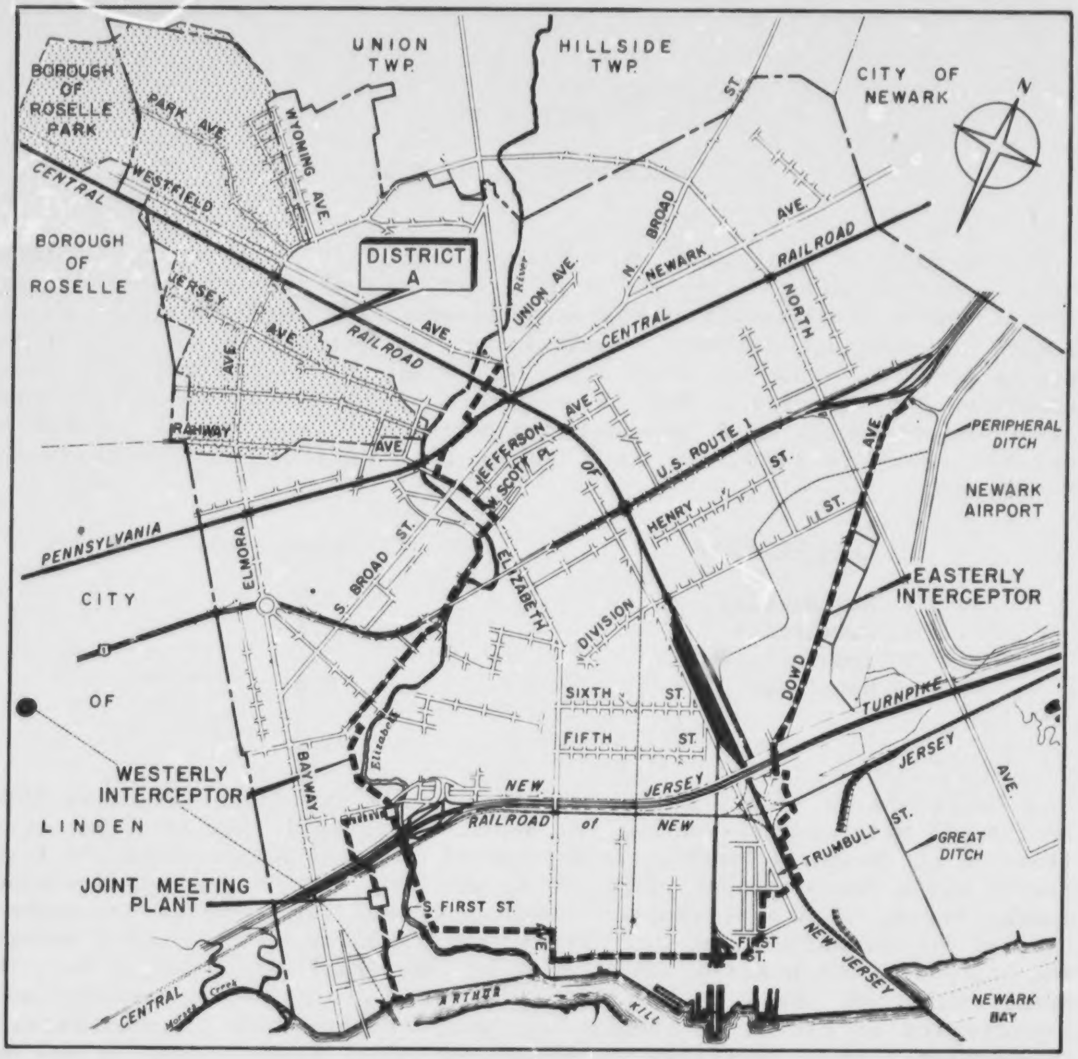

BOROUGH OF ROSELLE PARK
UNION TWP.
HILLSIDE TWP.
CITY OF NEWARK
N
PARK AVE.
WYOMING AVE.
WESTFIELD
CENTRAL
BOROUGH OF ROSELLE
DISTRICT A
JERSEY
AVE.
RAILROAD
River
UNION AVE.
N. BROAD
ST.
NEWARK
AVE.
CENTRAL
RAILROAD
NORTH
AVE.
RAHWAY
AVE.
JEFFERSON
AVE.
N. SCOTT PL.
OF
U.S. ROUTE 1
ST.
HENRY
ST.
AVE.
PERIPHERAL DITCH
NEWARK AIRPORT
PENNSYLVANIA
ELMORA
S. BROAD ST.
ELIZABETH
DIVISION
EASTERLY INTERCEPTOR
CITY
OF
SIXTH ST.
FIFTH ST.
DOWD
TURNPIKE
Elizabeth
BAYWAY
WESTERLY INTERCEPTOR
NEW
JERSEY
JERSEY
AVE.
LINDEN
RAILROAD of NEW
TRUMBULL ST.
GREAT DITCH
JOINT MEETING PLANT
S. FIRST ST.
FIRST ST.
NEW JERSEY
CENTRAL
Morses Creek
ARTHUR
KILL
NEWARK BAY

Figure 1. Study area, Elizabeth, New Jersey

Because the increased infiltration, as noted in other studies, is coincident with peak inflow rates, the amount of inflow treated in 1973 was only about 1.9 million m^3 (500 million gallons) per year. Assuming an average number of about 60 events per year during which combined sewage overflow occurs, the average quantity of inflow accepted to the Plant would be about 30,300 m^3 (8 million gallons) per event. The quantity of wastes discharged as untreated combined sewage during 1973, including the dry-weather Westfield Avenue discharge, is estimated at about 7.6 million m^3 (2.0 billion gallons). Approximately 80 percent of this discharge entered the Elizabeth River, with the remainder discharging to the Arthur Kill, Great Ditch, Peripheral Ditch and Newark Bay. Since such discharges can contain significant quantities of pollutants, including oxygen demanding substances, suspended solids and coliforms, they exert an adverse environmental effect on the receiving waterways. The inadequacy of the existing system also resulted in frequent flooding of streets and cellars with combined sewage. Hence, an environmentally effective solution for correction of the Elizabeth sewer system requires:

(1) a reduction of infiltration, and

(2) alleviation of the overflows and street and cellar flooding by combined sewage,

to the extent justified as cost-effective.

Detailed studies were made in Drainage District A (Figure 1) to develop criteria for water quality management for the entire City. The drainage area contains 265 hectares (655 acres); with 47 hectares (115 acres) located in Roselle Park; 2 hectares (5 acres) in Union and the remainder in Elizabeth. The area is largely developed for residential use (about 90 percent), with some neighborhood commercial (about 5 percent) and small industrial areas (about 3 percent). It includes most of the northwest region of the City and is served almost entirely by combined sewers. Dry-weather flow from most of the District is conveyed to the Westerly Interceptor which generally follows the course of the Elizabeth River. Part of the District, generally west of Elmora Avenue and south of Park Avenue, is served by the Joint Meeting Trunk Sewers, which traverse the District. Dry-weather flows and limited combined sewage flows are treated at the Joint Meeting Treatment Plant, located east of Bayway Avenue and north of South First Street in the City. Excess combined sewage flows discharge to the Elizabeth River. The existing sewer system in the District is badly undersized and six areas flood with combined sewage at almost every significant rainfall. A preliminary design has been prepared for a separate drainage system to relieve the overloaded combined system. This system provides for the drainage of the six frequently flooding areas plus other low spots, and for the separation of urban drainage and municipal wastes. A storm drain is not provided on every street, but reliance is placed, wherever possible without detrimental effects, on overland flow for the collection of urban drainage.

The population of the City is close to saturation and is expected to have only a moderate future growth. Inadequacies in the existing sewer system result in numerous complaints of street and basement flooding and of pollution in the Elizabeth River. Secondary treatment facilities are now under construction at the Joint Meeting Plant. These facilities are designed to provide secondary treatment for a nominal average flow of 3.3 m^3/sec (75 mgd) of which 0.9 m^3/sec (20 mgd) is assigned to Elizabeth. By installation of a low head effluent pumping station, the plant would, during wet weather, hydraulically provide secondary treatment for flows of up to 7.9 m^3/sec (180 mgd) and primary treatment for flows between 7.9 and 9.6 m^3/sec (180 and 220 mgd) at all tide elevations. The City has allocated to it, a peak wet-weather flow capacity of 1.8 m^3/sec (40 mgd). The Corps of Engineers has also planned a diked storage area, with a total capacity of about 80,000 m^3 (21 million gallons) along the Elizabeth River near the Joint Meeting Plant.

The Elizabeth River is tidal from its mouth at Arthur Kill for about 4,900 m (16,000 feet) to the Penn-Central Railroad. The river is small, draining only a total of 60 square kilometers (23 square miles) at its mouth. The analysis of the flow records at Westfield Avenue, about 5,500 m (18,000 feet) upstream from the mouth indicated the 7-day, 10-year return interval low flow to be 0.21 m^3/sec (4.8 mgd). Samples taken during slack periods at both low and high tides showed the river was polluted below the location of combined sewage overflow outlets.

SECTION VII

SUGGESTED METHODOLOGY

REQUIREMENTS TO BE MET

The need to evaluate the effects of:

1. random rainfall patterns and intensity,
2. the long-term and shock pollutional loads from combined sewage overflows,
3. varying amounts, modes of operation, and locations of storage,
4. varying interceptor and treatment capacity,
5. peak flow equalization, and
6. rainfall patterns on pollutant concentration

to develop cost-effective facilities for combined sewage overflow pollution abatement is apparent. This need arises from the following understanding of combined sewage pollution characteristics as developed for Elizabeth:

1. Approximately three out of four storms have peak rainfall rates during the last three quarters of the rainfall period. Such storms exhibit two peaks of mass pollutant washout. The first peak is of high-concentration, but low-flow volume, and the second of low-concentration, but high-flow volume peak.
2. The remaining one in four storms have the unusual pattern with peak rainfall rates occurring in the first quarter of the rainfall period. Such storms have only one peak with the maximum SS concentration about one order of magnitude less than the first peak of the usual storm pattern, but a substantially greater volume.
3. Most of the pollutant washout in the first peak of all the usual pattern storms, and in the frequent return interval, unusual pattern storms, results from solids deposition in the sewers during dry weather and is dependent upon the dry-weather sewage characteristics.

4. Significant amounts of street pollutant washout occur with more intense storms than those causing the severe pollutant discharge from sewer solids deposits. Urban runoff, therefore, is generally lower in pollutant concentration than the first flush of combined sewer flow.

5. Every one or two weeks, on an average, a rainfall can be expected to cleanse essentially all dry-weather solids deposits in sewers.

Hence, capturing for treatment combined sewage generated by ordinary rainfalls is the primary requirement for effective, combined sewage pollution abatement.

The variables that affect the pollutants contained in combined sewage overflow resulting from average rainfalls are many and complex. The most significant are

1. Time of day when rainfall occurs. Combined sewage generated by a rainfall between 3 and 6 a.m. will contain relatively small amounts of low-strength municipal sewage, (excepting pollution from dry weather flow solids deposit) and hence will be weaker than combined sewage generated by the same amount of rainfall between noon and 2 p.m., when the greatest amounts and higher strengths of municipal sewage are usually experienced.

2. Day of the week. Municipal sewage is usually stronger and of greater volume on weekdays than on Saturdays, Sundays and holidays.

3. Sewer slopes. The flatter the sewer slope, the lower will be the velocity of dry-weather flows and the more solids will be settled and later resuspended and discharged in the combined sewage overflows.

4. Rainfall pattern and intensity. The overflows from storms of frequent return interval contain a higher concentration of pollutants than do the greater storms.

5. Period since last rainfall. The amount of pollutants deposited in sewers increases with the duration of the antecedent dry period as does the street pollutant accumulation. There are large masses of data existing in the literature indicating pollutant concentration for various combined systems and rainfall amounts. Much of the available data has not been correlated, because the nature of the phenomenon may not have been fully understood, and all the necessary information may not have been obtained. (10,15)

The selection of a "design storm" or a series of "design storms" would not assess adequately many of the significant pollutional aspects of combined sewage overflows. Hence, development of a "design storm" for pollution control is not a viable approach. A long-term (12-year) continuous rainfall record and calculated runoff and quality characteristics, based on STORM default values, were used in this study to determine the relative effectiveness of alternative facilities for pollution abatement.

METHODOLOGY OUTLINE

A practical methodology has been developed in this study for the analysis required to determine cost-effective sewer systems in urban areas for combined sewage overflow pollution abatement. The following outlines the tasks of the suggested methodology:

1. Delineate the drainage districts of the study area and select a typical drainage district for detailed analysis.

2. Prepare data for the typical district for input to computer models.

   This would include:

   (a) The existing long-term (10-year) precipitation and air temperature data applicable to the area to account for the random nature of rainfall events.

   (b) Development of synthetic storms of 1-month, 6-month, 1-year and 5-year return intervals to determine overflow quantity and quality for the various alternatives for pollution control.

   (c) Subdivision of the typical drainage district and development of data such as land use, street length, population, sewer network and sizes, and street sweeping practice for input to SWMM.

   (d) Preparation of district-wide data (land use, population, area and street length, etc.) of all drainage districts to develop the total combined sewage flows for the City.

   (e) Development of dry-weather flow quantity and quality including infiltration.

   (f) Preparation of cost data, including costs of sewer pipes, storage, pumping stations, treatment, flushing, chemicals and regulators.

3. Evaluate alternatives in the typical drainage district using computer technology.

   This would include:

   (a) Field sampling and calibration of computer models for both quantity and quality of combined sewage.

(b) Analyzing the capacity of existing sewer systems.

(c) Establishing the characteristics of runoff quantity and quality from the existing system for various synthetic or real storm events.

(d) Using STORM and SWMM to evaluate the combined sewage overflows pollution reductions to be achieved by various alternatives, including:

1) upstream storage either in detention basins or sewers,

2) controlled routing in sewers,

3) sewer separation,

4) sewer rehabilitation,

5) overflow containment,

6) overflow treatment with and without containment, and

7) interceptor capacity upgrading.

(e) Comparing the cost-effectiveness of alternatives.

(f) Developing parameters of runoff quantity and quality (i.e., normalized hydrographs and pollutant loading for various land uses) for application to the remaining districts of the planning area.

4. Cost-effectiveness evaluation of alternatives for the entire planning area. The "Effectiveness" and "Efficiency" Indices may be used for scanning the various alternatives. The "Effectiveness Index" is obtained as follows:

$$\frac{\sqrt{\text{S.S.(mg/l)} \times \text{BOD(mg/l)}}}{\text{Average Overflow Return Interval (Days)}} \times \text{Annual Overflow Volume (MG)}$$

Physically, it represents the relative quantities of pollutant mass per day in the combined sewage overflows during overflow periods. The "Efficiency Index" is the product of the Effectiveness Index and the cost of the sewer system in dollar units. These parameters also permit development of the optimum level of treatment beyond which effectiveness diminishes markedly with increased cost increments.

RESULTS ATTAINED

The methodology develops, for the various alternatives, the average number of annual overflows, the annual volume of combined sewage overflows, the duration of the overflows, and the annual mass of BOD and SS discharged.

Combined sewage discharges are, by their nature, a severe shock load on the receiving waters.

The effectiveness of the various alternatives with respect to mitigating shock discharge loads are compared using the "Effectiveness Index". The logic for selecting the parameters which comprise this index follows:

1. The longer the return interval between overflows, the less should be the impact of such flows on the river system.

2. The impact on the river system would be proportional to the concentration of pollutants discharged thereto. Hence, the use of the geometric mean weights the effects of both SS and BOD.

3. The total volume of overflow would also affect the river quality.

The lower the value of the "Effectiveness Index", the less should be the impact of combined sewage overflows on the receiving water.

The "Efficiency Index" determines the most efficient alternative. The higher the "Efficiency Index", the less desirable is the alternative. The "Efficiency Index" is a measure of cost-effectiveness. The relationship of the "Efficiency" and "Effectiveness" Indices should also be compared for the various alternatives. If Alternative A has a lower "Effectiveness Index" but a higher "Efficiency Index" than Alternative B, it means the incremental work proposed to achieve Alternative A may be too expensive and other alternatives should be explored.

SUPPORTING APPENDICES

This report contains five appendices which provide background data for the methods used in this study. Appendix A presents the justification for using SWMM and STORM. Appendix B discusses modifications made to SWMM and STORM. Appendix C compares the differences in the two models with respect to determinations of quantity and quality. Appendix D describes, in detail, the required data for computer models, including meteorological data, the synthetic hyetograph development, sewer network configuration, hydraulic properties and land use of District A for both SWMM and STORM, discrete data, the area and land use characters for the remaining urban area of Elizabeth, dry-weather flow quantity, quality and characteristics, and basic cost data. Appendix E describes the method of calibrating the STORM runoff coefficient.

SECTION VIII

DESIGN AND COST COMPARISON OF SEWER SYSTEMS IN DISTRICT A

Designs of the three types of alternative sewer systems (conventional combined, conventional separate, and advanced combined with varying amounts of storage) were made for District A and respective costs estimated, assuming new sewer systems. The effectiveness of the alternative sewer systems in District A in controlling wet-weather flow problems (flooding and pollution) were also evaluated. New sewer systems were assumed in the evaluation to permit a uniform comparison and the transfer of the study findings to other districts within the City and to other urban areas.

The conventional separate sewer system consisted of two systems, one to carry wastes from domestic, commercial and industrial discharges together with such infiltration/inflow as must be anticipated, and the second to carry storm runoff. There are no storage facilities or flow regulators in either system. In the sanitary sewer system, almost every street contains a sewer, while in a storm sewer system, street gutter and overland flow to the maximum extent practical is allowed. Consequently, the total length of the storm sewer system is shorter than that of the sanitary sewer system. In newly developing areas, where ponding for urban runoff can be provided, storm sewer sizes can also be significantly reduced.

In a conventional combined system, sewers receive both municipal wastewater and stormwater. In this study, by definition, flow control devices and/or storage facilities were not provided in the collection system for control of combined sewage flow rates and quality. Collection sewers (including laterals and trunks) were used for conveyance of both dry-weather and wet-weather flows. Combined sewage flow rates in excess of the capacity of the intercepting sewers (which carry flow to storage/treatment facilities) overflow to receiving waters. A combined sewer system and a separate sanitary sewer system have the same total length. As street runoff normally enters inlets located at the downstream end of a street, the sewer in the upstream reach of a sewer network branch would usually carry sanitary wastewater only. Consequently, the length of sewers carrying combined sewage is less than the total length of a separate sanitary system. In the case of District A, about 40 percent of the length of sewers in the combined system would accept only sanitary sewage. Modification of the combined system which permitted the same amount of overland flow as the separate system was also evaluated.

The advanced combined sewer systems were designed to provide various amounts of storage for wet-weather flow control. Two types of storage were considered: in-pipe storage and satellite storage. In in-pipe storage systems, sewers are increased in size to provide volumetric storage in addition to flow conveyance. Regulators are installed to permit use of in-pipe storage to reduce peak flows, thus increasing the amount of polluted runoff intercepted and later treated for equal interceptor capacities. In-pipe storage has been successfully implemented in the existing combined sewer systems of Seattle, Detroit, Cleveland and Minneapolis-St. Paul. The storage may be incorporated in upstream lateral sewers or downstream trunk sewers or both. In this study, in-pipe storage was found to be more economically located in upstream lateral sewers, as will be described more fully hereinafter. There is only about half the volume in the lateral sewers as in the larger size trunk sewers in a conventional combined system, and the additional storage for both pollution abatement and flood control may be obtained more economically by enlarging smaller size rather than larger size sewers.

In a satellite storage system, collection sewers are designed for flow conveyance, but detention basins are located in the collection sewer system to reduce peak flows. There would be little volume available in the sewers for storage because of the small volume in the laterals and reduced size and volume in the trunks. The detention basins could be open, covered or tunneled structures. In the literature, "satellite storage" is frequently referred to as "off-line storage". In this study, however, the term "off-line storage" is reserved for flow equalization basins located near the sewage treatment plant. In both the conventional and advanced combined systems, off-line storage may be provided. Satellite storage can be constructed at any location within a drainage district where there is sufficient flow to use it effectively. With respect to the city-wide sewer network, satellite storage should be viewed as upstream as opposed to off-line or downstream storage. There could be several satellite storage structures at strategic locations in each of the 25 drainage districts in Elizabeth. The number of off-line storage basins would be much fewer. As both in-pipe and satellite storages are located in the collection sewer system, both will be generally termed as "in-system" storage.

Since the basic difference between an advanced and a conventional sewer system is the in-system storage provided, the sewer layout and total sewer length of the two systems are identical. However, the pipe diameters differ significantly.

DESIGN PHILOSOPHY

The selection of the rainfall event or events on which to base the design of a combined or separate system for urban runoff is of major importance, and has, too often, been only casually addressed. The traditional approach has been to:

1. develop the intensity-duration-frequency curves for the location under study. These are normally available from U.S. National Weather Service Technical Paper Nos. 25 and 40 (37,38), or other such sources.

2. select a return interval for which it is desired to provide flood protection. Typically, a return interval of two, five or ten years has been used.

3. develop a design storm hyetograph from the intensity-duration curve for the selected return interval. This storm is a synthetic rainfall event which may be based on meteorological probabilities or may be an envelope storm which contains within a single event the depth-duration characteristics of all naturally occurring events likely within the chosen return interval (9).

4. Use the intensity-duration curve directly in situations where a synthetic storm hyetograph is not developed, e.g., when the analysis of the system is to be performed using the Rational Method. This use has similar characteristics as an envelope event.

During the past five years, as use of relatively powerful computer-based models such as SWMM has grown, users continued to develop the required "design storm" input in the manner described above. However, the increased capabilities of the model compared to hand-computation techniques has led to numerous questions concerning the "shape" of the design storm and the appropriateness of any single storm for the design of the system. The usual questions are related to the following items:

1. Where should the peak of the "design storm" be located? The intensity-duration-frequency curves do not include any information on the "shape" characteristics of naturally occurring rainfall events.

2. What antecedent conditions should be used?

At the same time as these issues were being raised, the developemnt of the various Water Quality Control Acts has forced the designer to consider quality as well as quantity in the design of a storm sewer system. Only recently has it been appreciated that the return interval appropriate for quantity control is not appropriate for the elimination of the typical pollution-causing discharge. The runoff quality from any event is largely a function of the pollutant which has accumulated in the sewer and on the overland surface since the last rainfall event. The following aspects of storm selection now have been recognized:

1. Rainfall rates of about 25.4 mm/hr (1.0 in./hr) are sufficient to washoff about 90% of the pollutant from the surface (6). Slightly lower rates will be sufficient for paved surfaces, while higher rates may be required to cause significant erosion from non-urban areas. Substantially lower rates should flush out most sewer solids deposits.

2. Rainfall depths of between 5.1 and 12.7 mm (0.2 and 0.5 inches) are sufficient to washoff most of the accumulated pollutants from a

typical urban area.

3. Since the available pollutant is largely a function of the intervals between rainfall events, the distribution of storms within the typical year is significant.

The majority of the polluting overflows result from small events (as compared to the traditional design event). Hence, the system must be able to handle the hydraulic design storm without excessive flooding, while, at the same time, it must be capable of regulating the frequent rainfall runoff which, if allowed to discharge freely, would result in major pollution. Several models such as STORM were developed to allow analysis of the performance of the proposed/present system under long-term rainfall behavior. These models employ simplified hydraulics, and little or no sewer network data, and can not adequately estimate the response of the system to the large storm event. Rather, they serve as a mechanism for evaluating the type of control strategies which should be employed.

The availability of both SWMM and STORM provides the mechanisms to evaluate both the "day-to-day" and "rare-design-storm" characteristics of this system. However, the inputs to the models must be both compatible and representative of the rainfall characteristics of the area. For the present study, the following inputs were adopted:

1. The continuous hourly precipitation data at the Newark International Airport gauge was used as input to the STORM program. This location is quite close to the study area, and its rainfall characteristics are quite similar. Rainfall data for the 12-year period from 1963 to 1974 was used, and was assumed to be uniform within the study area.

2. A 5-year return interval intermediate patterns envelope type storm hyetograph was used as the design criteria for flood protection for the City of Elizabeth. The intensity-duration curves for Sandy Hook, New Jersey, were used as a basis for the synthetic storm generation. An analysis of the 12-year Newark Airport data used, indicated close agreement between the intensity-duration curves for Newark and Sandy Hook. However, the latter was used for the development of the hyetograph since they represent an analysis of a much longer period of record than the 12-year Newark data.

It is felt that this combination of rainfall inputs can confidently be used to design a system which is capable of controlling the pollutional aspects of typical daily or weekly flows, while at the same time providing runoff control capabilities sufficient to handle the hydraulic flows of the 5-year event.

The synthetic hyetograph, as developed in Appendix D, was used to design alternative facilities for evaluation. This type of hyetograph was conceived in 1959 by the senior author after investigation of failures resulting with drainage structures designed using meterological hyetograph (9) or other methods. The total rainfall precipitation represented in this hyetograph at any elapsed time should about equal the historical total for that time interval, and the total volume of runoff may be expected to equal, within reasonable limits, the total for the interval at the stated return period interval. However, the actual rainfall intensities for shorter increments within the selected time interval may be expected to vary from, and be less than, those assumed in the synthetic hyetograph. Hence, the peak flows experienced will probably be less than flows computed with the hyetograph.

The magnitude of the difference between the flow rates obtained using the synthetic rainfall hyetograph and the probable 5-year peak runoff flows is influenced by the short time of travel for peak runoff flows from the upstream reaches to the overflow point. Previous studies in Elizabeth (8) determined that, for 93 drainage areas containing less than 49 hectares (120 acres) each, the time of travel was less than 12 minutes. For District A, the time of travel is about 20 minutes. Such short times of travel are frequently found in cities with populations of about 100,000 or less. Based on analyses by others (9), the short increment rainfall rates of 5-year storms of various durations would be as follows, when expressed as a multiple of the increments of a 360-minute duration rainfall.

Time Increment (minutes)	Total Rainfall Duration (Minutes)				
	5	30	60	180	360
5	1.37	1.22	1.16	1.06	1.0
10	-	1.20	1.15	1.05	1.0
15	-	1.18	1.14	1.04	1.0
20	-	1.18	1.13	1.05	1.0
30	-	1.16	1.11	1.05	1.0
60	-	-	1.10	1.04	1.0
180	-	-	-	1.03	1.0

Hence, at the point where a system would be stressed by a 5-minute peak rainfall rate, computations using the synthetic hyetograph, which satisfies the intensity-duration curve at each time interval, would be about 30 percent greater than that obtained from the peak 5-minute flow from a 180-minute duration storm. By extrapolation, the following relations with respect to the 180-minute storm were developed for reaches which would be stressed by other time intervals.

Time Interval (min.)	% Greater Flow from Synthetic Hydrograph
10	27
15	24
20	21
25	18
30	15

It is improbable that a 5-year return frequency storm of duration equal to the most severe flow generating time for each sewer reach would be experienced every five years. Hence, the synthetic hyetograph chosen may generate peak flow rates that are, perhaps, 15 to 30 percent greater than those experienced, although the total runoff volume required for design of pollution abatement facilities should be about right. As demonstrated hereinafter, the use of the 180-minute duration hyetograph should permit determination of the runoff volume required to develop pollution abatement facilities.

The design philosophy and methodology developed in this study is based on a synthetic hyetograph, which reasonably establishes runoff volumes required for effective pollution abatement for design of alternatives. The methodology permits determination of the effects of a wide variation in storage volumes on the design of facilities. The alternatives developed are tested using a long-term continuous record of real rainfall events to determine their effectiveness for abatement of pollution from combined sewage overflows. No attempt is made to develop a design storm or storms with respect to pollution abatement.

CONVENTIONAL COMBINED SEWER SYSTEM

Design

The new conventional combined sewers in District A were designed using SWMM with capacity to convey runoff from a 5-year return interval, intermediate pattern synthetic storm. The combined sewer system layout is shown in Figures D-8 to D-12. The total length of the system is 32,308 m (106,000 feet), of which 19,397 m (63,640 feet) carry combined sewage and 12,911 m

(42,360 feet) carry sanitary sewage. The sewers carrying sanitary sewage are all 0.2 m (8 inches) diameter.

The total length of lateral sewers conveying combined sewage (those pipe elements numbered less than 300) is 15,651 m (51,350 feet) and of trunk sewers 3,746 m (12,290 feet). There are 139 designated lateral sewers conveying combined sewage and 31 trunk sewers in District A, with pertinent pipe elements shown in TABLE D-7. These sewers receive runoff from 279 subcatchments with subcatchment characteristics tabulated in TABLE D-6.

SWMM allows routing rainfall runoff in sequence from overland through lateral sewers and trunk sewers to the outfall. Lateral sewers were sized with the SWMM RUNOFF Block and trunk sewers with the SWMM TRANSPORT Block. The design diameters of the lateral sewers for the combined system are shown in TABLE 1 and of the trunk sewers in TABLE 2, under 0 percent in-pipe storage. Design diameters for the advanced combined system are also shown for comparison and later discussion.

Two sets of design diameters are shown in TABLE 1 (laterals) and TABLE 2 (trunks), for the combined system. The lateral sewers were designed with two methods of flow routing. In both cases, the hydrographs of the 5-year synthetic storm from tributary subcatchments to lateral sewers (gutter and pipes) are identical and are computed with the SWMM RUNOFF Block (up to Subroutine WSHED). In computing the set of design diameters for lateral sewers not in parentheses, the design flow was the peak of the inflow hydrograph developed by adding the inflow hydrographs of upstream pipes and local tributary subcatchments using a subroutine derived for this study. The design flow, without reduction due to pipe storage, was assumed to enter the upstream end of the pipe. Manning's equation was used to compute the required pipe diameter for full flow condition and the next largest commercial size was used as the design diameter. This method was developed to permit determination of peak flow with in-pipe storage as later described. The lateral sewer sizes in parentheses were computed with the Subroutine GUTTER of the SWMM RUNOFF Block which uses Manning's equation to determine maximum pipe flow together with storage routing using the continuity equation. Where only numbers not in parentheses are shown, there is no difference in pipe diameter under the two methods.

Sewers sized assuming no storage routing or time lag in the hydrograph are generally at least as large as those sized considering storage routing or time lag of hydrograph. Of the 139 lateral sewers considered, 111 have the same sewer diameter and 19 designed with the first method are greater and 9 are smaller in diameter than those designed with the second method. However, some significant reductions are found in element numbers 210, 225, 239, 237 and 219, which are the larger diameter and more costly pipe reaches.

With respect to in-pipe storage pipe sizes, discussed later, pipe element number 105 decreases in diameter with increased total volumes of in-pipe storage. This results because of the moderating effect of outflow from pipe element number 49 and no tributary area directly drains to it. Pipe element number 77 is the minimum size used in a combined sewer and

TABLE 1. LATERAL SEWER DESIGN DIAMETERS (inches)

Pipe Element No.		In-pipe Storage Volume**			
	0%*	8.3%	16.5%	25.0%	41.3%
82	27	60	84	102	126
101	36	48	60	72	90
99	30	48	60	66	84
98	27	48	60	72	90
95	36 (42)	36	42	42	54
94	36	42	54	60	72
92	42 (48)	48	60	66	84
91	36 (30)	42	48	60	72
189	48	48	60	66	84
187	36 (40)	65	90	108	132
85	36	48	60	66	84
104	21	42	60	72	90
103	27	36	48	54	72
108	36	48	60	72	90
112	36 (30)	48	60	66	84
119	30	48	60	72	90
78	36	36	42	48	60
80	36	42	54	60	72
83	30 (36)	24	21	18	18
121	27	48	60	72	90
123	30	42	54	60	78
125	36	42	54	60	72
126	42	30	27	24	18
84	42	60	78	96	120
128	36	54	66	78	96
148	27	36	42	48	60
149	18	24	36	42	48
147	21	42	54	66	84
146	36	60	72	90	114
143	48	60	72	84	108
141	48	60	78	90	114
139	60	66	78	84	108
138	21 (18)	30	42	48	60
135	48	60	72	84	102
87	48	42	48	54	60
89	54	60	66	78	96
133	54	48	48	54	60
1	27	48	60	72	90
3	30 (36)	48	66	78	96
4	27	54	66	78	102
7	18	36	48	54	66
8	24	36	48	54	66

NOTE:

* Conventional Combined Sewer System. Numbers in the parenthesis are pipe sizes obtained from SWMM and should replace the numbers not in parenthesis as the combined sewer sizes.

** Advanced Combined Sewer System. Storage volumes as percent of 119,000 m^3 (31.5 million gallons) (runoff volume of the 5-year synthetic storm for District A).

TABLE 1. (continued)

Pipe Element No.	In-pipe Storage Volume**				
	0%*	8.3%	16.5%	25.0%	41.3%
10	27 (30)	48	60	72	90
13	27 (24)	48	60	72	90
14	30	36	42	54	60
15	21	42	54	60	78
20	27	42	54	60	78
19	30	48	60	66	84
24	36 (30)	48	60	72	84
25	21	36	48	60	78
29	24	42	60	66	84
30	36	54	72	84	108
33	27	48	66	78	102
31	36	48	66	78	96
35	30	24	21	18	18
42	21	42	54	66	78
40	27	42	60	66	84
38	30	42	54	66	84
36	48	48	54	60	72
47	24	42	54	60	78
49	27 (24)	36	42	48	60
56	27	42	54	66	84
52	36	54	66	78	96
54	36	42	48	54	56
55	42	42	54	60	72
114	42	36	27	24	21
117	27	42	54	66	84
59	36	42	48	54	66
72	27	42	60	66	84
74	27	54	72	84	108
75	36 (30)	36	42	54	60
67	27 (24)	42	60	66	84
105	27 (24)	21	18	18	18
109	36	48	54	66	84
129	21	24	27	30	42
77	18	18	18	18	18
76	21 (18)	36	42	48	60
71	36	36	42	42	54
69	42	60	72	84	108
64	54	54	60	66	84
62	54	54	54	60	72
60	54	50	72	78	102
58	48	42	48	48	60
201	36	84	114	138	174
202	54	54	60	72	84
203	54	96	132	156	192
204	54	48	48	54	60
205	54	60	72	84	108
206	54	54	60	66	84
207	42	42	48	48	60
208	30	48	60	72	84
212	42	60	84	96	120
213	42	42	48	54	66
214	48	60	78	90	114
215	27	54	66	78	102

TABLE 1. (continued)

Pipe Element No.	In-pipe Storage Volume**				
	0%*	8.3%	16.5%	25.0%	41.3%
216	36	42	48	54	66
209	48 (54)	48	48	54	66
210	60 (54)	54	54	60	72
211	48	42	48	54	60
220	30	48	66	78	96
221	36	60	84	96	126
227	27 (24)	30	36	42	54
226	30	48	54	66	84
225	42 (36)	54	66	78	96
228	27 (24)	42	54	60	78
229	30	54	72	84	108
230	36	36	42	48	54
233	30	60	78	90	114
222	36 (42)	66	84	102	126
232	42	48	60	66	78
235	24	36	48	60	72
234	27	30	36	42	48
231	42 (36)	54	66	78	102
224	48	36	30	27	24
223	66	60	60	66	72
236	24	36	48	54	66
238	27 (30)	60	78	90	120
246	24	36	48	60	72
249	27	42	54	66	84
251	27	42	60	66	84
253	27	42	54	66	84
252	36	48	60	72	90
250	42	42	48	54	60
248	48	36	36	42	42
247	54	54	60	66	84
245	60	60	66	72	84
244	24	36	48	60	72
243	66	60	66	78	90
242	54	60	66	78	96
241	54	60	72	84	102
240	54	60	72	84	102
239	60 (54)	48	54	60	66
237	66 (60)	60	72	78	96
260	36	54	72	84	108
261	36	54	66	78	96
262	54 (48)	54	66	78	96
263	21	30	42	48	60
264	36	60	78	90	120
219	102 (96)	84	78	78	84

NOTE:

* Conventional Combined Sewer System. Numbers in the parenthesis are pipe sizes obtained from SWMM and should replace the numbers not in parenthesis as the combined sewer sizes.

** Advanced Combined Sewer System. Storage volumes as percent of 119,000 m^3 (31.5 million gallons) (runoff volume of the 5-year synthetic storm for District A).

TABLE 2. TRUNK SEWER DESIGN DIAMETERS (FEET)

Pipe Element	Conventional Combined System	Advanced Combined System						
		In-pipe Storage *				Satellite Storage *		
Number	0%	8.3%	16.5%	25.0%	41.3%	9.4%	17%	41.3%
301	3.00	2.25	2.00	1.75	1.50	2.75	2.75	2.75
303	4.00 (3.5)	3,00	2.75	2.50	2.00	3.50	3.50	3.50
305	4.50 (4.0)	3.50	3.00	2.75	2.25	4.00	4.00	4.00
307	4.50	4.00	3.50	3.00	2.50	4.50	4.50	4.50
309	5.50 (5.0)	4.00	3.50	3.50	2.75	5.00	5.00	5.00
311	6.50 (6.0)	5.00	4.50	4.00	3.50	5.50	5.50	5.50
313	6.50 (6.0)	5.00	4.50	4.00	3.50	6.00	6.00	6.00
315	7.00 (6.5)	5.50	5.00	4.50	4.00	6.50	6.50	6.50
317	8.00 (7.0)	6.50	5.50	5.00	4.50	8.00	7.50	7.50
319	6.50 (6.0)	5.00	4.50	4.50	4.00	6.00	6.00	6.00
321	9.00 (8.0)	7.50	6.50	6.00	5.50	8.00	8.50	5.00
323	8.50 (8.0)	7.00	6.50	6.00	5.00	7.50	8.00	4.50
325	9.00	7.50	6.50	6.00	5.00	8.50	8.00	5.00
327	6.50	5.50	5.00	4.50	4.00	6.50	6.00	4.00
329	7.50 (8.0)	6.50	6.00	5.50	4.50	7.50	6.50	5.00
331	9.00	8.00	7.00	6.50	5.50	9.00	7.50	6.00
333	6.50	5.50	5.00	4.50	4.00	6.50	5.50	4.50
335	7.50	6.50	5.50	5.00	4.50	7.00	6.50	5.50
337	7.50	6.50	5.50	5.00	4.50	7.00	6.50	5.50
339	9.00 (9.5)	8.00	7.00	6.50	5.50	8.50	7.50	7.00
341	10.00	8.50	7.50	7.00	6.00	9.50	8.50	7.00
343	10.00	9.00	8.00	7.00	6.50	10.00	8.50	8.00
345	10.00	9.00	8.00	7.50	6.50	9.50	8.00	8.00
347	13.50	12.00	11.00	9.00	9.00	13.00	11.00	10.00
349	14.00 (13.0)	12.00	11.00	9.50	9.00	12.50	11.50	11.00
351	14.50 (14.0)	11.50	12.00	9.50	9.50	12.50	11.50	10.50
353	12.50	11.00	10.50	9.00	8.00	11.50	10.50	9.50
357	10.50 (11.0)	9.50	9.00	7.50	7.00	10.00	9.00	8.00
359	10.50 (11.0)	10.00	9.00	7.50	7.00	10.00	9.00	8.00
361	10.50 (11.5)	10.00	9.00	8.00	7.00	10.00	9.00	8.00
363	11.00 (11.5)	10.00	9.00	8.00	7.00	10.50	9.00	8.00

* Storage volumes are in terms of percent of a 5-year design storm runoff volume, 119,000 m^3 (31.5 million gallons) for District A.

is not reduced in size as a result of moderated outflows from pipe element number 129. Pipe element number 219 is reduced in size with in-pipe storage because of the moderating effects of outflow from upstream pipes.

A similar comparison of trunk sewer sizes for the conventional combined system is given in TABLE 2. The difference in sewer sizes, with four exceptions, is no more than one commercial size and is due to differences in hydrographs entering the trunk sewer inlet points since the same TRANSPORT Block was used to size the diameters. Of the 31 trunk sewer conduit elements considered, 13 have the same sizes. Twelve associated with the first method are greater and six are smaller than those associated with the second method. However, the increases are in the larger size pipe although the largest and second largest size pipes (element numbers 349 and 351) are reduced in size. The difference in diameters sized with two design methods reflects the small pipe storage effect for a 5-year synthetic storm. Those diameters obtained with subroutine GUTTER of SWMM (as modified) were used as the combined sewer sizes.

The initial diameters specified for each conduit element when using SWMM for conduit sizing affect the final design diameters as well as the outflow hydrograph and pollutograph. This results since SWMM does not recompute the previously computed portion of the hydrograph when pipes are resized due to the occurrence of surcharge at a later time in the design storm. Generally, the effect on the design diameter is small but could be significant on the peak outflow hydrograph.

The trunk sewer decreases as the amount of in-system storage increases. This results from the reduced peaks of the outflow hydrographs from either satellite storage or in-pipe storage. This characteristic is a significant factor in determining the least-cost system for both pollution and flood control.

Outflow Hydrographs and Pollutographs

The 5-year return interval synthetic storm and outflow hydrograph from District A for the conventional combined system is shown in Figure 2. The peak outflow rate is 39.8 m^3/sec (1,407 cfs). A 2-minute integration interval was used. The outflow hydrograph maintains a relatively low value until about one hour after the rainfall starts. The time lag between the peak of the hyetograph and the peak of the outflow hydrograph is about 12 minutes, indicating a short response time of the drainage system to the synthetic storm. The time of travel in the trunk sewers, estimated assuming all pipes flowing full, is about twice as long as the time required for the peak flow to be generated at the downstream outlet.

As discussed previously, the initial diameters specified as input to SWMM affect the design diameters and the flow routing. Hence, the outflow hydrograph and pollutographs are influenced. For example, Case A of TABLE 3 specified initial pipe diameters to be about one-half of the final design diameter and results in a peak hydrograph of value 44.4 m^3/sec (1,569 cfs). In Case B, initial diameters were assumed to be one or two commercial

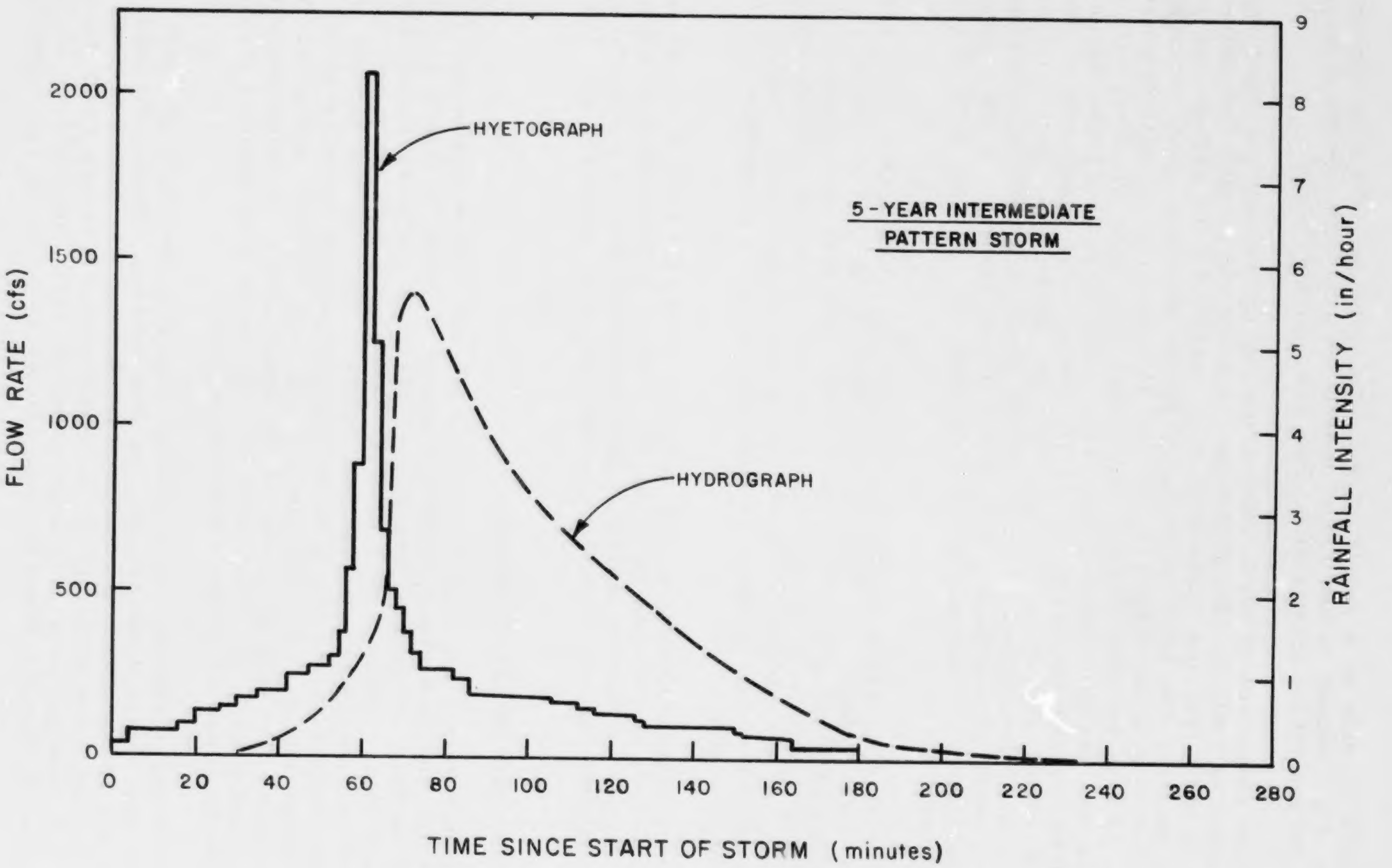


Figure 2. Hyetograph and outflow hydrograph, District A (5-year intermediate pattern storm)

TABLE 3. COMPARISON OF DESIGN DIAMETERS AND PEAK HYDROGRAPHS

Trunk Sewer No.	Case A		Case B		Case C	
	(1)	(2)	(1)	(2)	(1)	(2)
301	1.5	3.0	2.5	3.0	3.0	3.0
303	1.5	4.0	3.5	3.5	3.5	3.5
305	1.5	4.0	3.5	4.0	4.0	4.0
307	1.5	4.5	4.0	4.5	4.5	4.5
309	2.0	5.0	4.5	5.0	5.0	5.0
311	2.0	6.0	5.5	6.0	6.0	6.0
313	3.0	6.5	6.0	6.0	6.0	6.0
315	3.0	7.0	6.5	6.5	6.5	6.5
317	3.0	7.5	7.0	7.0	7.0	7.0
319	2.5	6.0	5.5	6.0	6.0	6.0
321	4.0	8.5	8.0	8.0	8.0	8.0
323	4.0	8.0	7.5	8.0	8.0	8.0
325	4.0	8.5	8.0	9.0	9.0	9.0
327	3.0	6.5	6.0	6.5	6.5	6.5
329	3.0	7.5	7.0	8.0	8.0	8.0
331	4.0	9.0	8.5	9.0	9.0	9.0
333	3.0	6.5	6.0	6.5	6.5	6.5
335	3.0	7.5	7.0	7.5	7.5	7.5
337	3.0	7.5	7.0	7.5	7.5	7.5
339	4.0	9.0	8.5	9.5	9.5	9.5
341	4.0	10.0	9.5	10.0	10.0	10.0
343	4.0	10.0	9.5	10.0	10.0	10.0
345	5.0	10.0	9.5	10.0	10.0	10.0
347	4.0	12.0	12.0	13.5	13.5	13.5
349	4.5	13.0	13.0	13.0	13.0	13.0
351	4.5	14.0	13.0	14.0	14.0	14.0
353	6.0	12.5	11.0	12.5	12.5	12.5
357	5.0	11.0	10.5	11.0	11.0	11.0
359	5.0	11.0	10.5	11.0	11.0	11.0
361	5.0	11.0	10.5	11.5	11.5	11.5
363	5.0	11.5	10.5	11.5	11.5	11.5
Peak hydrograph flow	1569 cfs		1509 cfs		1407 cfs	

Note: (1) Input diameters in feet
(2) Design diameter in feet

1 foot = 0.3048 meters
1 cfs = 0.0283 m^3/sec

sizes less (using Case A as a guide) than the final design diameters. The final design diameters were generally the same as or one commercial size different from those of Case A and the peak outflow hydrograph value from District A was reduced from 44.4 to 42.7 m^3/sec (1,569 to 1,509 cfs). In Case C, the initial pipe diameters were specified to be equal to the final design diameters of Case B. Sewer routings were consistently made throughout the duration of the synthetic storm. Because of proper sewer routing of design storm flow, the peak flow rate was reduced from 42.7 to 39.8 m^3/sec (1,509 to 1,407 cfs). The hydrographs corresponding to Case C of TABLE 3 were used for analysis.

Figures 3 and 4 present the outflow pollutographs in pounds per minute from the combined sewer system for SS and BOD, respectively. They assume four dry days prior to the occurrence of the storm. The street sweeping interval assumed was seven days (as now practiced in Elizabeth) and the sweeping efficiency 75 percent. STORM default values were used in computing pollutant constituents (SS, BOD) from the amount of dust and dirt in the street washout.

Figure 3 illustrates the two flushes in a combined sewer system. The first flush ends about 50 minutes from the start of the storm and the second flush, about 40 minutes later. The first flush reflects the deposit of solids in combined sewers from sanitary wastes during dry days and their resuspension in the initial phase of the storm. TABLE 4 shows the amount of solids deposited in each trunk sewer segment in District A during four antecedent dry days as computed with the SWMM TRANSPORT Block. The average daily solids deposit is about 127 kg (281 pounds). Most of these deposits occur in pipe element numbers 347, 349 and 351, which have the relatively flat slope of 0.0004. The solids deposits in these three elements account for about 84 percent of the total. This indicates the advantage of maintaining flushing velocities for dry-weather flows. A rainfall with an intensity of about 1/16 of the 5-year synthetic storm and with a return frequency of less than one month (about ten days) should cleanse these deposits in the sewers. The peak runoff rate from this small, frequent rainfall is 0.76 m^3/sec (27 cfs) as compared to 39.8 m^3/sec (1,407 cfs) for the 5-year design storm. This flow rate is about ten times the average dry-weather flow.

Figure 4 presents mass rate BOD pollutographs associated with the 5-year synthetic storm for both the separate storm and combined systems. The double peak, observed in the SS pollutograph, is not as apparent since, in the SWMM used, solids deposited in sewers are assumed to contribute only to SS and not to BOD. Such contribution should be considered based on the anticipated organic content associated with all sewer solids.

Had the number of antecedent dry days been more than four, the first peak of the mass rate SS pollutograph would be increased since solids deposit in combined sewers would become a dominant factor in total runoff polutants. The pollutograph, shown in Figure 5, assumes 30 antecedent dry days. The BOD for either the combined or separate system would mainly result from street washout as shown in Figure 6 by the similarity of pollutographs for both the combined and separate systems.

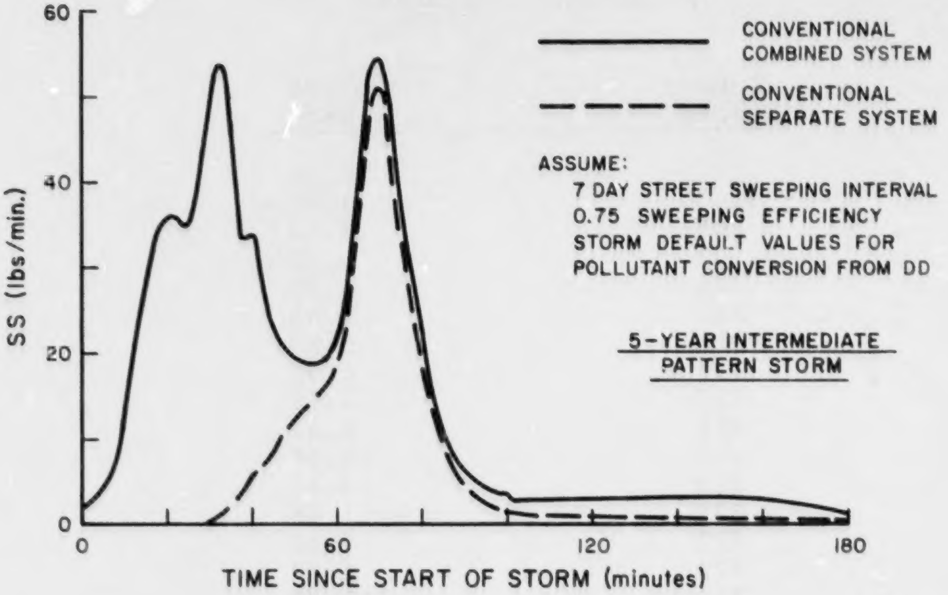


Figure 3. Mass rate SS pollutographs, 4 dry days (5-year intermediate pattern storm)

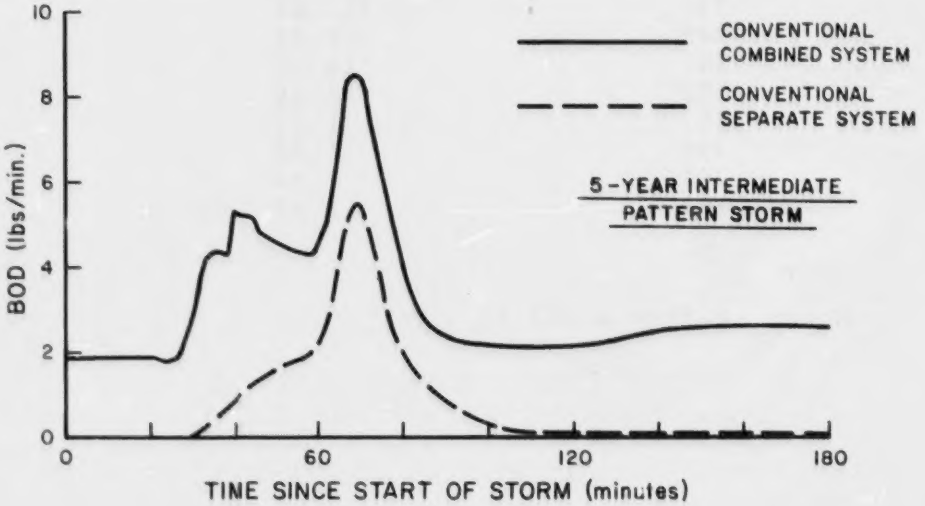


Figure 4. Mass rate BOD pollutographs, 4 dry days (5-year intermediate pattern storm)

TABLE 4. SOLIDS DEPOSIT (LBS) IN SEWER WITH FOUR ANTECEDENT DRY DAYS

Element Number	Solids (lbs)
301	6.48
303	4.88
305	3.52
307	3.61
309	4.23
311	5.65
313	5.87
315	6.53
317	7.18
319	0.37
321	15.36
323	12.41
325	11.66
327	0.40
329	0.67
331	12.78
333	0.29
335	0.52
337	0.51
339	2.60
341	13.80
343	13.76
345	13.75
347	311.61
349	306.40
351	322.76
353	28.69
357	1.73
359	1.86
361	1.92
363	1.92

Note: 1 lb = 0.453 kg

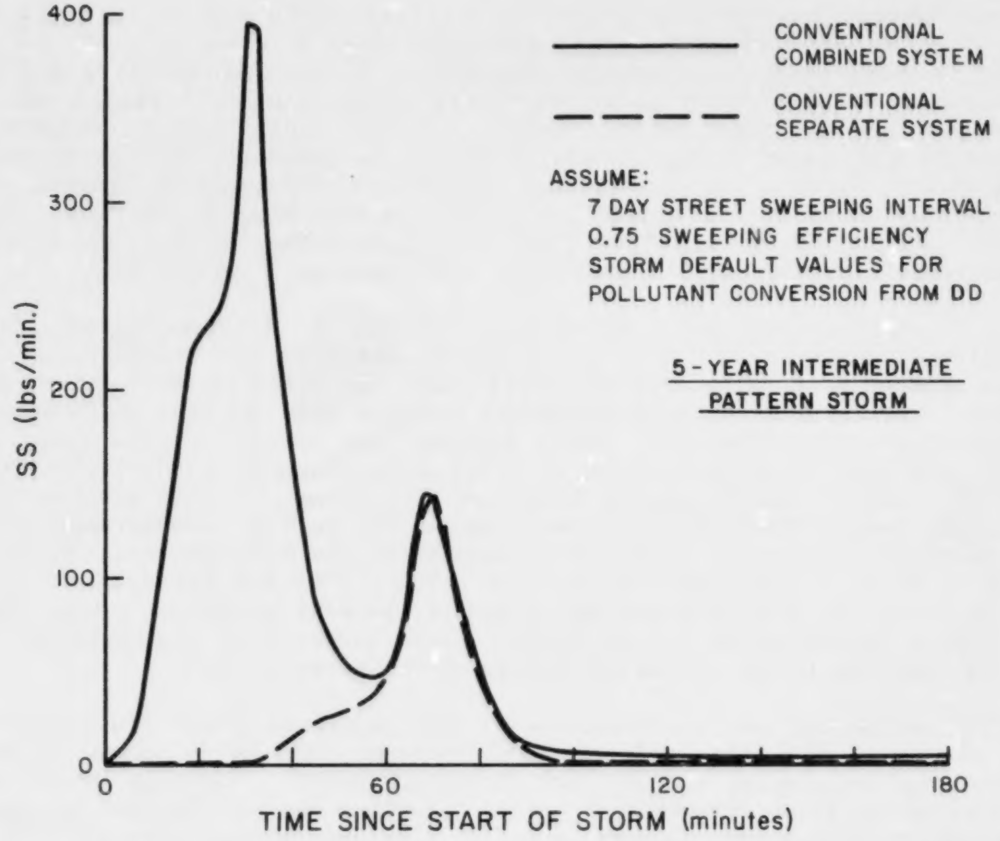


Figure 5. Mass rate SS pollutographs, 30 dry days (5-year intermediate pattern storm)

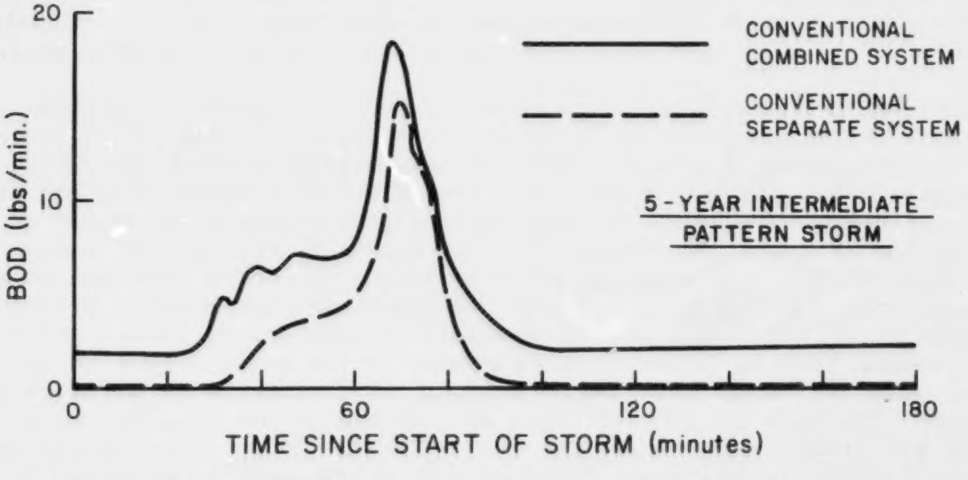


Figure 6. Mass rate BOD pollutographs, 30 dry days (5-year intermediate pattern storm)

To control the initial flush of pollution from solids deposits in the sewers, it would be necessary to store and treat a volume of about 1.6 mm (0.063 inches) over the drainage basin for a 5-year synthetic storm of intermediate pattern. The volume requiring storage is small since most of the solids are deposited in the larger and more flatly sloped downstream reaches of the sewer system. This condition is generally typical in most urban areas. If the solids are distributed more uniformly throughout the system, the volume to be stored could be larger. If the pollutants from street washout are to be controlled, a runoff volume of 23.0 mm (0.906 inches) should be stored and treated for the 5-year synthetic storm.

In terms of pollutant concentration (mg/l), a different pollutograph character is observed. Figure 7 shows the concentration pollutographs of the 5-year synthetic storm for BOD and SS for both the conventional combined and separate storm systems. The pollutograph contains only one peak which occurs early in the storm when the runoff quantity is small. In the combined system, the peak SS concentration of 1,972 mg/l occurs 18 minutes after the start of rainfall and 38 minutes later (or 56 minutes after the start of the storm) the concentration drops below 20 mg/l. The peak SS concentration from the separate storm system occurs 50 minutes after the storm starts. The peak has a relatively low value of 17 mg/l using STORM default values. The BOD concentration pollutograph has a similar general character except that the peak of pollutograph occurs earlier since there is no consideration of the BOD fraction in the solids deposit in storm sewers in SWMM.

The pollutographs associated with the separate storm sewer system were obtained using the combined sewer network and design diameters but without the DWF inputs to the SWMM TRANSPORT Block. Although there are differences in sewer network and design diameters between the two systems, the resulting differences in outflow hydrographs and pollutographs are within the range of accuracy required in this study for the comparison of the alternative systems.

Figures 8 and 9 present the outflow hydrographs and pollutographs of 1-year and 1.3-month synthetic storms, respectively. They are similar to those of the 5-year synthetic storm except in flow and pollutant magnitude.

Runoff characteristics from storms of advanced pattern were also investigated. Figure 10 shows the District A outflow hydrograph and pollutographs for a 5-year return interval storm with the advanced pattern hyetograph shown in Figure D-5. The peak runoff rate from District A occurs about 18 minutes (as compared to 74 minutes for intermediate pattern storm) after the start of the storm with a peak rate of 21.7 m^3/sec (766 cfs), about one-half of the corresponding intermediate pattern storm. The peak combined sewage SS concentration (138 mg/l) is one order of magnitude less than the peak concentration (1,972 mg/l) for the intermediate pattern storm. This results since peak runoff from an advanced storm occurs in the early stages of the storm event when the results of sewer flushing is combined with street washout. However, the peak SS concentration of stormwater runoff alone is 92 mg/l, about five times higher than the peak (17mg/l) for an intermediate pattern storm. There is one peak flush of pollutant mass in advanced pattern storm runoff. The peak flush ends about 45 minutes after the storm starts.

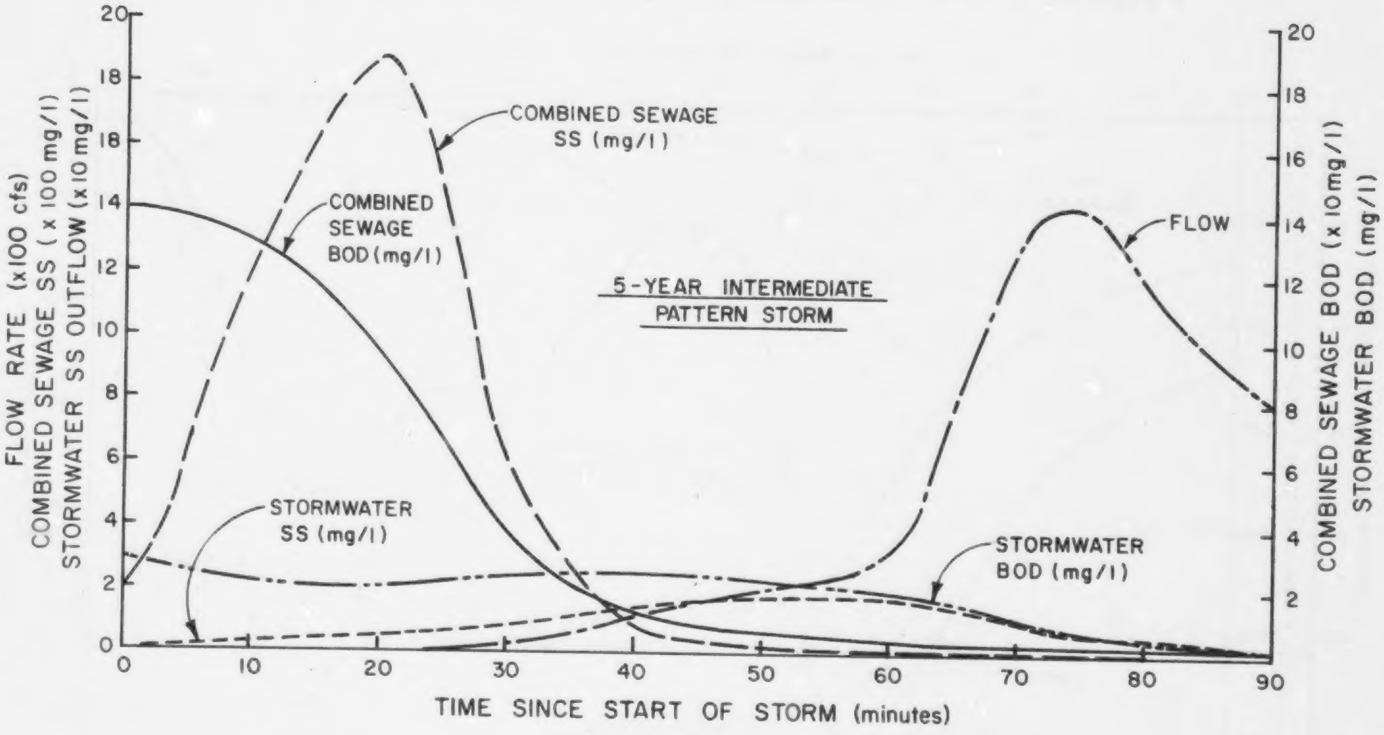


Figure 7. SS, BOD concentration pollutographs, 5-year intermediate pattern storm

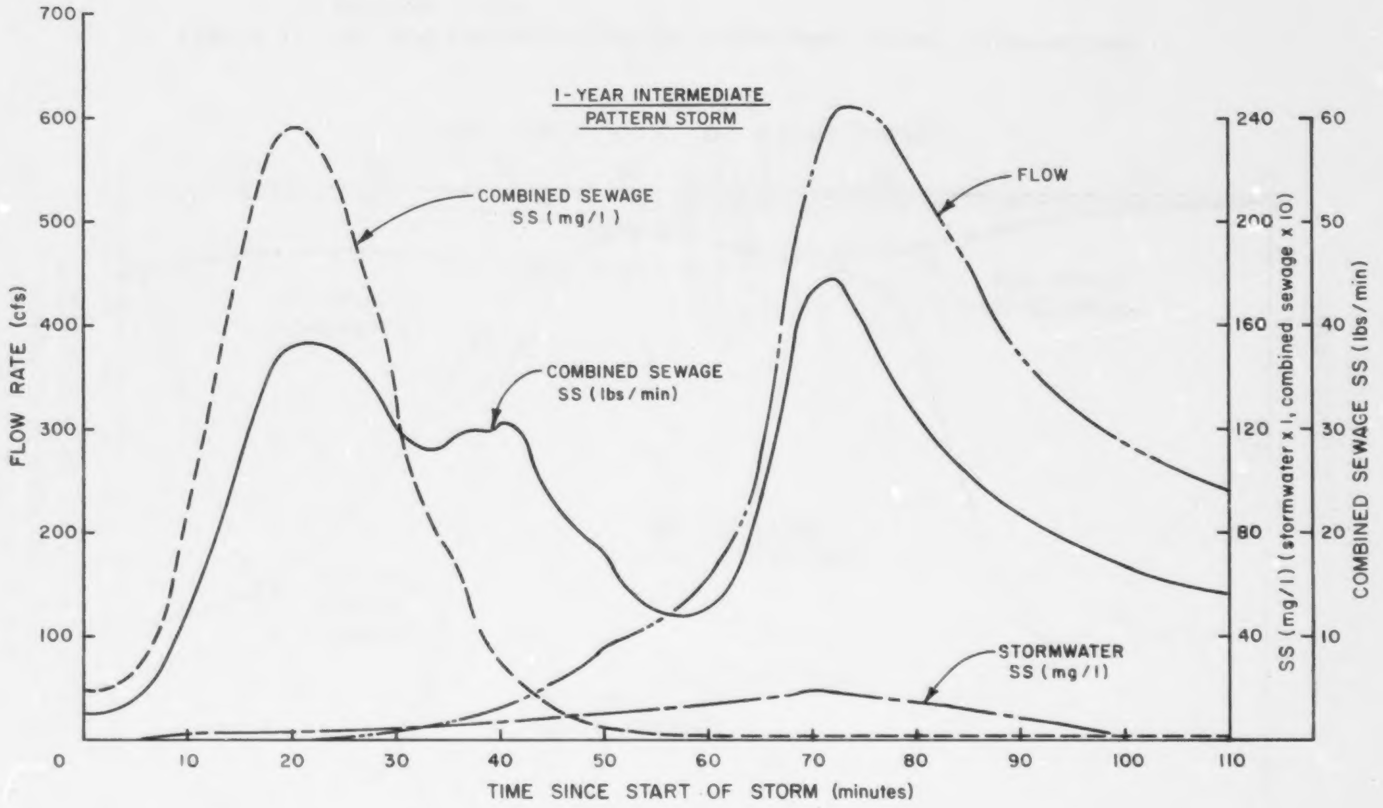


Figure 8. Outflow hydrograph and pollutographs, 1-year intermediate pattern storm

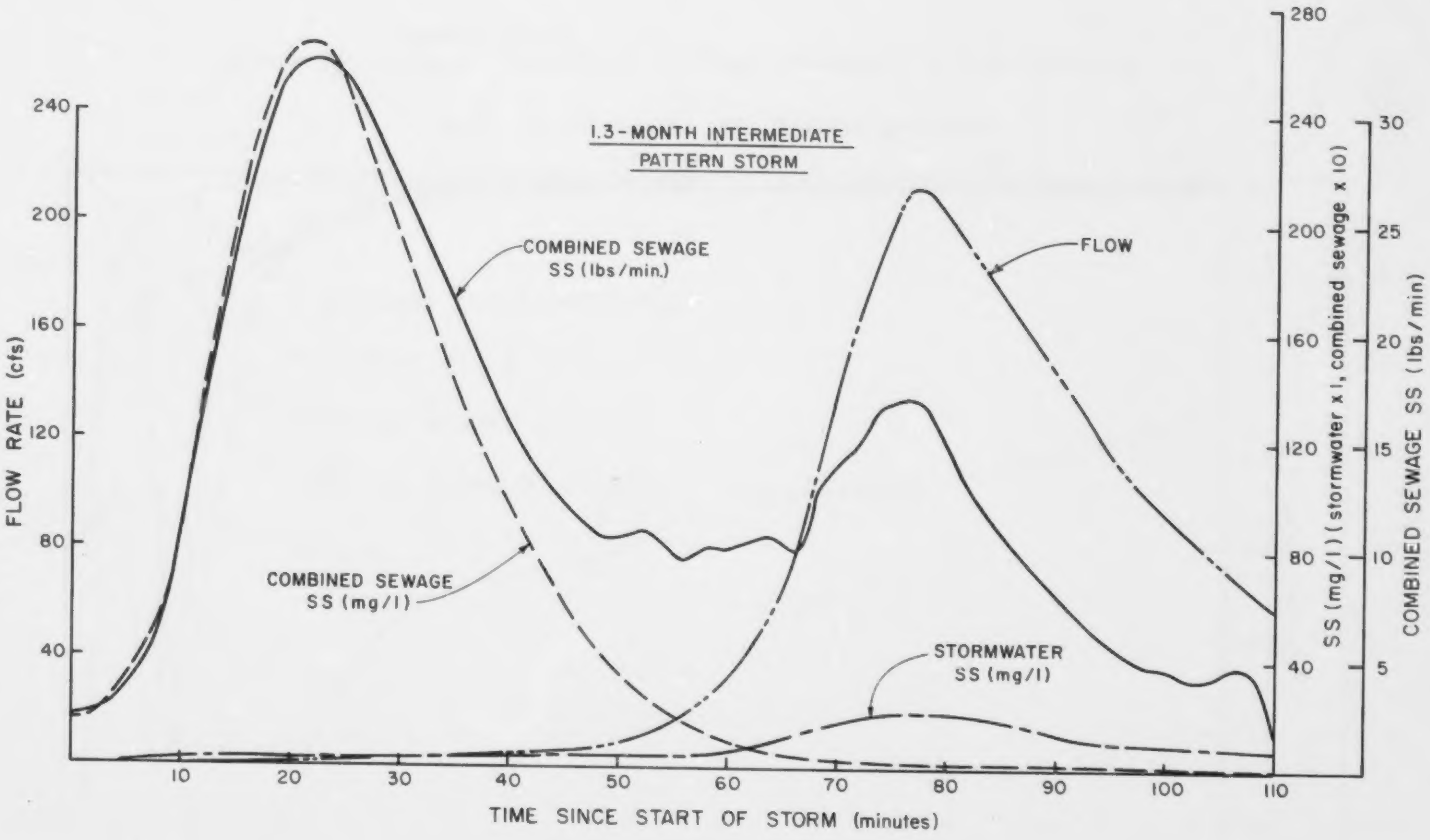


Figure 9. Outflow hydrograph and pollutographs, 1.3-month intermediate pattern storm

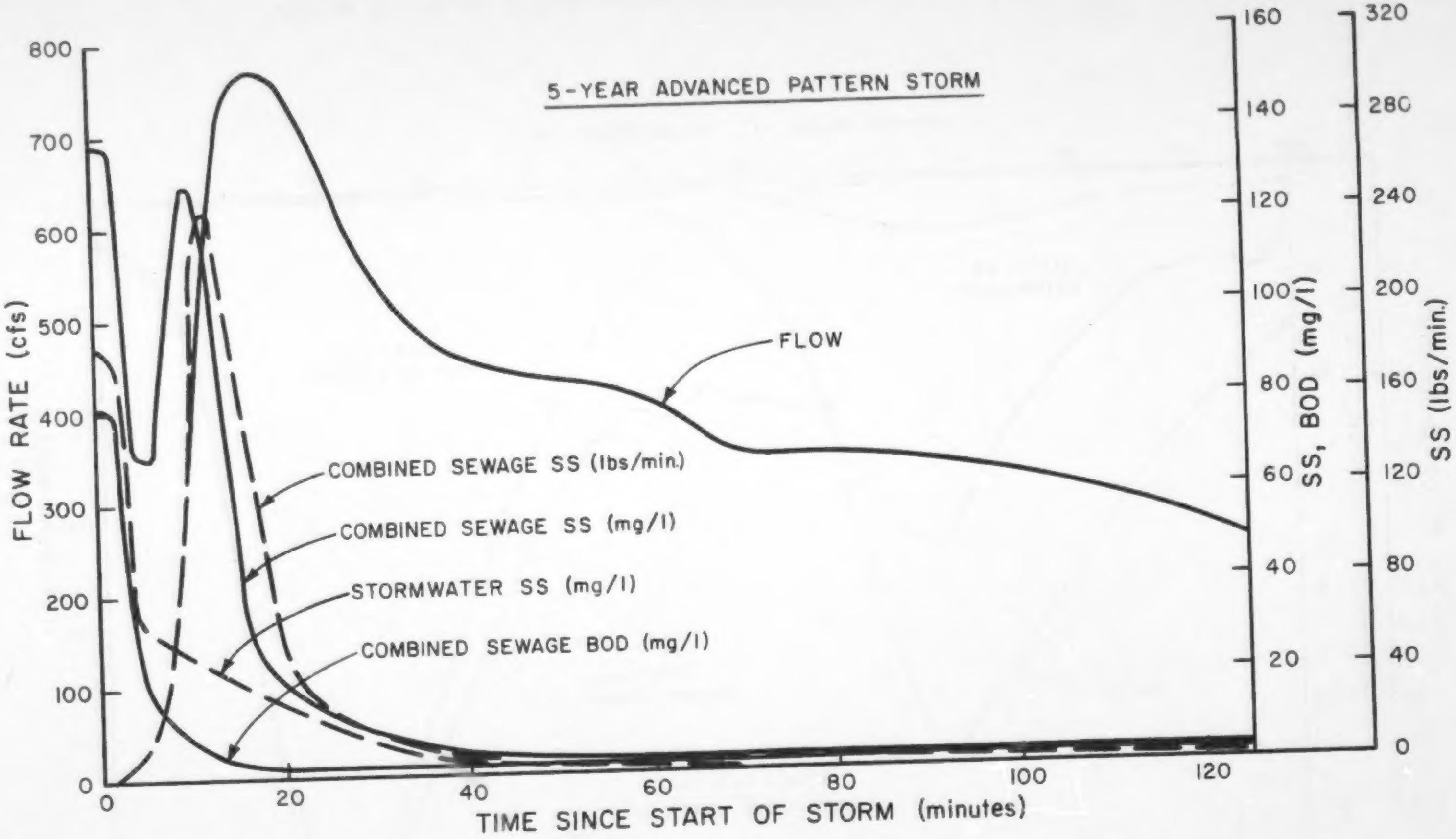


Figure 10. Outflow hydrograph and pollutographs, 5-year advanced pattern storm

Characteristics of runoff from 1-year and 1.3-month synthetic storms are similar to those of the the 5-year synthetic storm except for magnitude of runoff rate and pollutant amounts.

Although runoff characteristics between storms of intermediate and advanced patterns are substantially different, differences appear insignificant in the total pounds of pollutants reaching the outlet of the sewer system as shown in TABLE 5. The computed SS loadings in District A combined system are 1,331 and 1,329 kg (2,939 and 2,933 pounds), respectively, for the synthetic 5-year intermediate and advanced pattern storms. For a separate storm system, the computed values are 506 and 500 kg (1,116 and 1,104 pounds), respectively. For BOD, the values are 321 versus 320 kg (709 versus 707 pounds) for a combined sewer system, and 58 versus 57 kg (128 versus 126 pounds) for a storm sewer systems. However, the peak concentrations for the advanced pattern are much lower than for the intermediate pattern due to the dilution afforded to the first flush.

The following is noted from TABLE 5:

(1) For a given rainfall amount, both the runoff volume and peak runoff rate from an intermediate pattern storm are greater than those from an advanced pattern storm with the exception of the runoff volumes for 1.3-month return frequency storms, which are equal, since most of the runoff from this storm is from impervious areas.

(2) Frequent, smaller storms, regardless of pattern, generally produce higher pollutant concentrations than infrequent, rare storms. This is illustrated in Figures 11 and 12, which, respectively, compare the SS concentration pollutographs versus storm return frequencies for the combined and separate storm sewer systems in District A. Conclusions from BOD concentration pollutographs are similar.

(3) Rains with a return interval of about 10 days (0.35 months) (intensity 0.1 inches/hour) should cleanse combined sewers. The total SS loading reaching the drainage district outlet from this storm is computed as 827 kg (1,826 pounds), of which 325 kg (718 pounds) is estimated to be from DWF. The remaining pollutant mass is attributable to the sewer solid deposits, as the street washout from this small storm would be insignificant. During the four dry days assumed in the computation, the total amount of solids deposit is estimated to be 509 kg (1,124 pounds).

(4) The total SS accumulation on the streets of District A, assuming four antecedent dry days, a 7-day street sweeping interval, and 75 percent sweeping efficiency, is computed as 525 kg (1,160 pounds). A 5-year synthetic storm would remove about 96 percent of the total, a 1-year synthetic storm, 87 percent, and a 1.3-month synthetic storm, 35 percent. Corresponding synthetic advanced storms would remove slightly less pollutant.

Effective pollution control in an urban area associated with wet-weather

TABLE 5. SUMMARY OF RUNOFF QUANTITY AND QUALITY CHARACTERISTICS FOR VARIOUS STORMS

		Quantity			Quality*							
					Peak Concentration (mg/l)				Total Loading (lbs)			
					Combined Sewage		Stormwater Runoff		Combined Sewage		Stormwater Runoff	
Storm Frequency	Storm Pattern	3-hour precipitation (inches)	Runoff Volume (inches)	Peak Outflow Rate (cfs)	SS	BOD	SS	BOD	SS	BOD	SS	BOD
5-Year	Intermediate	2.43	1.76	1407	1972	140	17.0	2.5	2939	709	1116	128
1-Year	Intermediate	1.41	0.82	616	2364	140	19.2	2.3	2827	693	1011	117
1.3-Month	Intermediate	0.60	0.26	215	2660	140	21.0	2.6	2215	631	406	60
0.35-Month**	Intermediate	0.15	0.05	27	2805	140	-	-	1826	577	-	-
5-Year	Advanced	2.43	1.52	766	138	78	92.	4.4	2933	707	1104	126
1-Year	Advanced	1.41	0.64	385	164	118	45	6.0	2683	680	864	103
1.3-Month	Advanced	0.60	0.26	130	448	135	-	-	2099	622	-	-

* Based on four antecedent dry days, seven days street sweeping interval, 75% sweeping efficiency and STORM default values for pollutant computation.

** Frequency estimated by extrapolation

- Data not available

Note: 1 inch = 25.4 mm; 1 cfs = 0.0283 m^3/sec; 1 lb = 0.0453 kg

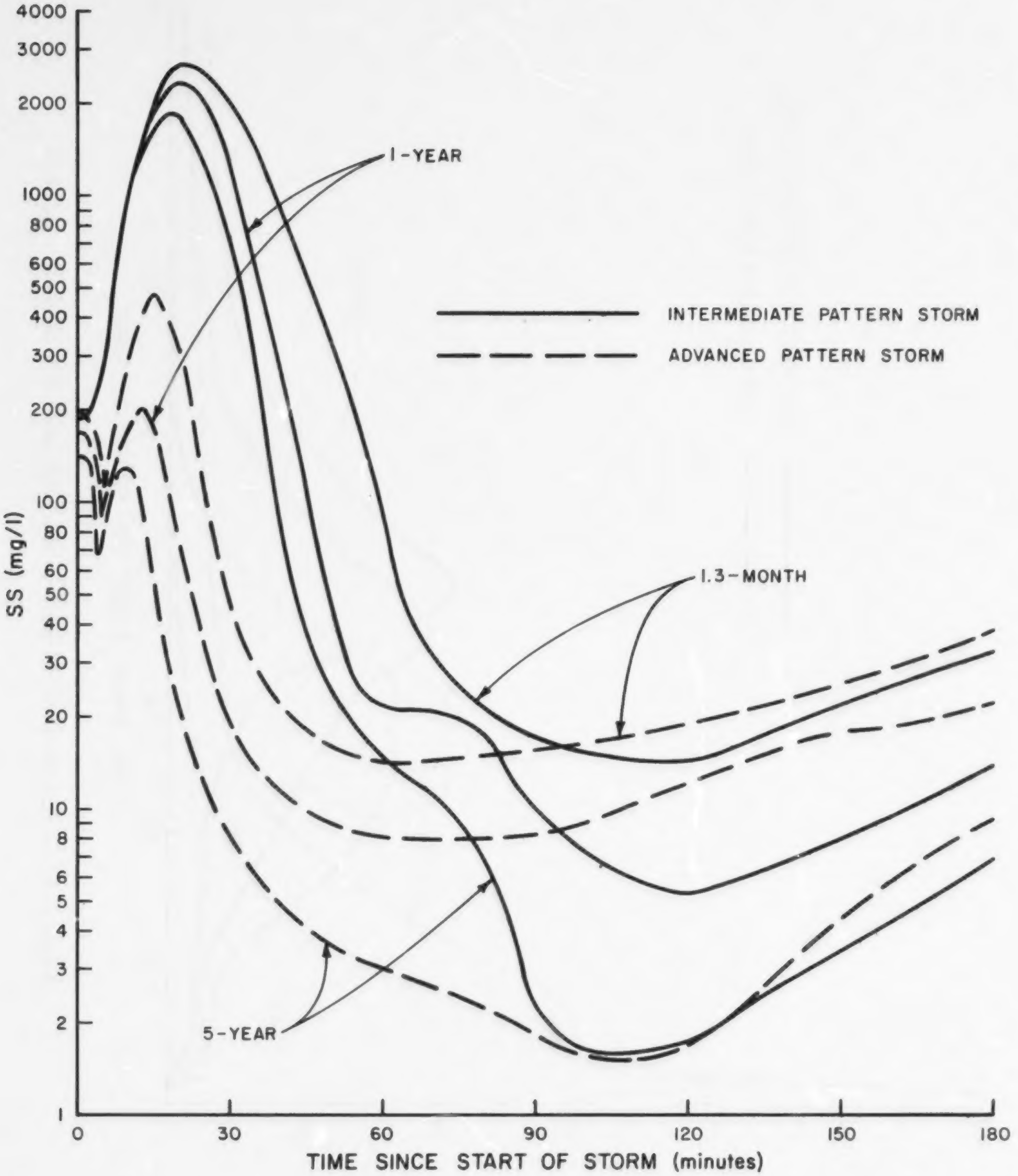


Figure 11. Combined sewage pollutographs for various storms

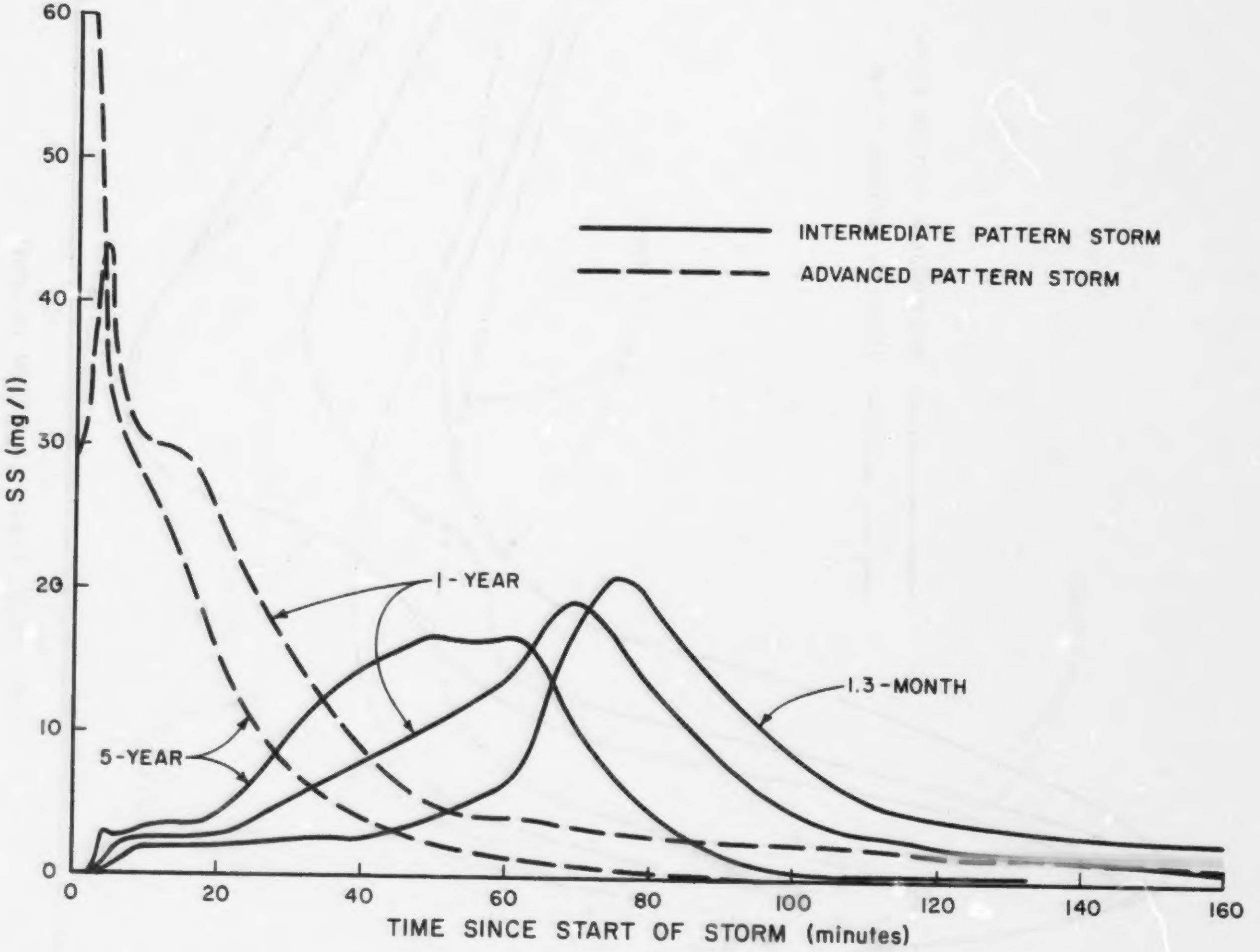


Figure 12. Stormwater runoff pollutographs for various storms

flows can be largely achieved by the containment and treatment of the first flush from frequently occurring storms.

SEPARATE STORM SEWER SYSTEM

In 1972, a separate storm sewer system was designed for District A (11). The existing combined sewers were to be maintained as a separate sanitary sewer system accepting limited amounts of urban runoff. The storm sewer layout is shown in Figure D-13. Its function was to relieve combined sewage flooding by intercepting runoff. The new combined sewer layouts were extensions of the separate storm system. Storm sewer slopes, diameters and average excavation depths for the trunk sewer elements in the system are listed in TABLE 6. Also listed is the corresponding information for the combined sewer system. The effect of eliminating surcharges in combined sewer systems is apparent. The storm sewer slopes were determined from detailed topographic surveys and field inspection of underground utilities to eliminate interference. The average excavation depth for each pipe element is computed taking into account the upstream and downstream pipes.

The separate storm sewer system was designed with surcharged sewers to reduce sizes and excavation cost. The following was used to determine the flows and hydraulic grade in the system. Hydrographs for both impervious and pervious areas were developed from the 5-year synthetic storm hyetograph. These hydrographs were determined for a 0.405 hectare (1.0 acre) area and a ground slope of one percent, by subtracting surface infiltration, depression storage and surface detention from the rainfall, for each minute of the storm. The flow in any particular section of the trunk sewer at any minute was determined by time routing the incremental volumes from upstream hydrographs. For each minute of the storm, the flow in each section of the trunk was successively determined. For trial pipe sizes, the hydraulic gradient was determined for each minute of the storm based on flowing full friction and secondary losses calculated for each section of trunk sewer. The pipe sizes selected were those which raised the peak hydraulic gradient to about one foot below the drain inlet grating at the lowest point in the system. This would still allow design flow from the critical low points to simultaneously enter the system. Different trunk sewer sizes necessitated reevaluating the time of travel in each section for proper time routing.

TABLE 7 compares peak flows at several locations of District A generated for the storm sewer design and for the design of combined sewer system using SWMM. The peak flow rate used in the separate storm sewer design is generally ten percent less than the flow used in the design of combined sewers to reduce the frequency of combined sewer surcharging. This results since the critical factor in a surcharged system is the hydraulic gradient. The peak hydraulic gradient in any sewer reach, except the most downstream reach, is influenced by backwater effects and occurs after the peak flow has passed downstream.

TABLE 6. COMPARISON OF PIPE DIAMETER, SLOPE, AND EXCAVATION DEPTH, SEPARATE STORM VS. COMBINED SYSTEMS

Trunk Sewer Element	Separate Storm System			Combined System		
	Diameter (feet)	Slope	Excavation Depth (feet)	Diameter (feet)	Slope	Excavation Depth (feet)
301	3.0	.0025	9.0	3.0	.0025	9.2
303	3.5	.0025	9.6	3.5	.0025	9.8
305	4.0	.0025	9.8	4.0	.0025	10.0
307	4.5	.0025	10.4	4.5	.0025	10.6
309	5.0	.0025	10.9	5.0	.0025	11.0
311	6.0	.0025	12.2	6.0	.0025	12.7
313	6.0	.0025	14.3	6.0	.0025	14.3
315	6.5	.0025	13.5	6.5	.0025	14.4
317	7.0	.0013	14.4	7.0	.0025	14.8
319	7.0	.0010	13.7	6.0	.0090	13.3
321	7.0	.0013	13.4	8.0	.0013	15.4
323	7.0	.0017	13.5	8.0	.0017	15.4
325	7.5	.0020	14.2	9.0	.0025	16.2
327	7.5	.0090	13.6	6.5	.0099	13.1
329	7.5	.0090	14.0	8.0	.0048	14.5
331	7.5	.0005	14.8	9.0	.0019	16.0
333	7.5	.0100	14.1	6.5	.0133	13.8
335	7.5	.0094	15.3	7.5	.0089	16.5
337	7.5	.0094	16.3	7.5	.0091	17.4
339	8.0	.0034	17.0	9.5	.0034	23.1
341	8.0	.0034	16.4	10.0	.0019	22.8
343	8.0	.0088	17.6	10.0	.0019	22.4
345	8.0	.0088	18.7	10.0	.0019	20.8
347	10.0	.0004	19.9	13.5	.0004	22.4
349	10.0	.0004	22.6	13.0	.0004	24.7
351	10.0	.0004	25.9	14.0	.0004	29.0
353	11.0	.0013	28.0	13.0	.0013	30.2
357	11.5	.0003	28.5	11.0	.0030	28.5
359	11.5	.0003	29.5	11.0	.0030	29.5
361	11.5	.0003	32.3	11.5	.0030	32.8
363	11.5	.0003	31.0	11.5	.0030	31.5

Note: 1 foot = 0.3048 meters

TABLE 7. PEAK FLOW COMPARISON FOR SEPARATE STORM AND COMBINED SEWER DESIGN (5 YEAR RETURN FREQUENCY)

Trunk Sewer Inlet No.	Drainage Area (Acres)	Peak Flow (cfs) Separate Storm System	Combined System
338	349.6	766	898
342	360.0	795	909
356	600.4	1259	1321
364	655.1	1276	1407

Note: 1 acre = 0.405 hectare
1 cfs = 0.0283 m^3/sec

There are noted differences of sewer sizes for the conveyance of stormwater runoff and combined sewage as shown in TABLE 6. The differences result from allowing surcharged flow in the separate sewer system and not in the combined system under design flows. All other conditions being similar, design diameters for a combined sewer system are generally equal to or larger than those for a separate storm system.

There is little difference in runoff hydrographs for the separate storm and combined system since dry-weather flow quantity is a small fraction of total runoff during wet weather. For instance, the peak 1.3-month synthetic storm runoff rates are 6.08 and 5.95 m^3/sec (215 and 210 cfs), respectively, for the combined and separate storm system.

SEPARATE SANITARY SEWER SYSTEM

The layout of the separate sanitary sewer system (Figures 13 and 14) generally follows the existing sanitary or combined sewers, except that the existing three outlets have been reduced to two. Each city street is served by one sanitary sewer. There are 260 sewer pipe elements with a total length of about 32,308 m (106,000 feet).

The quantity of domestic sewage was determined using the estimated saturation population and the estimated daily wastewater flow per capita. For District A, the total saturation population was estimated at 16,500 and the average daily domestic flow was calculated as 4.34 m^3 per min. (0.65 mgd), assuming a rate of 379 liters (100 gallons) per capita per day including infiltration. The ratio of peak flow to average daily flow was estimated using the formula suggested by Harmon (12)

$$\frac{M}{Q} = 1 + \frac{14}{4 + \sqrt{P}} \qquad (1)$$

where:

M = the instantaneous peak flow,
Q = the average daily domestic flow, and
P = the tributary population in thousands.

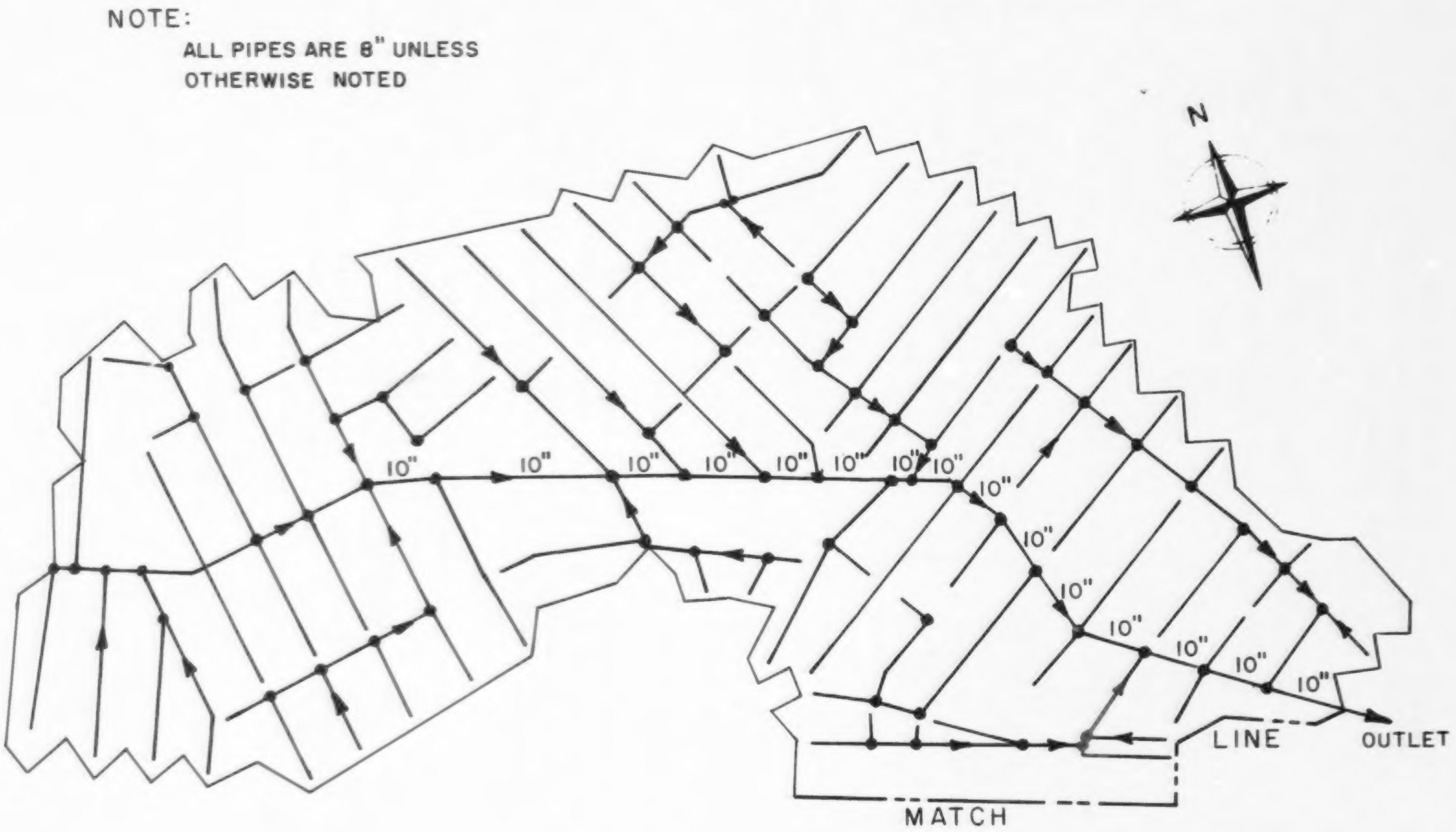

NOTE:
ALL PIPES ARE 8" UNLESS
OTHERWISE NOTED
N
10"
10"
10"
10"
10"
10"
10"
10"
10"
10"
10"
10"
10"
10"
10"
10"
OUTLET
LINE
MATCH

Figure 13. Sanitary sewer layout, subareas I and II

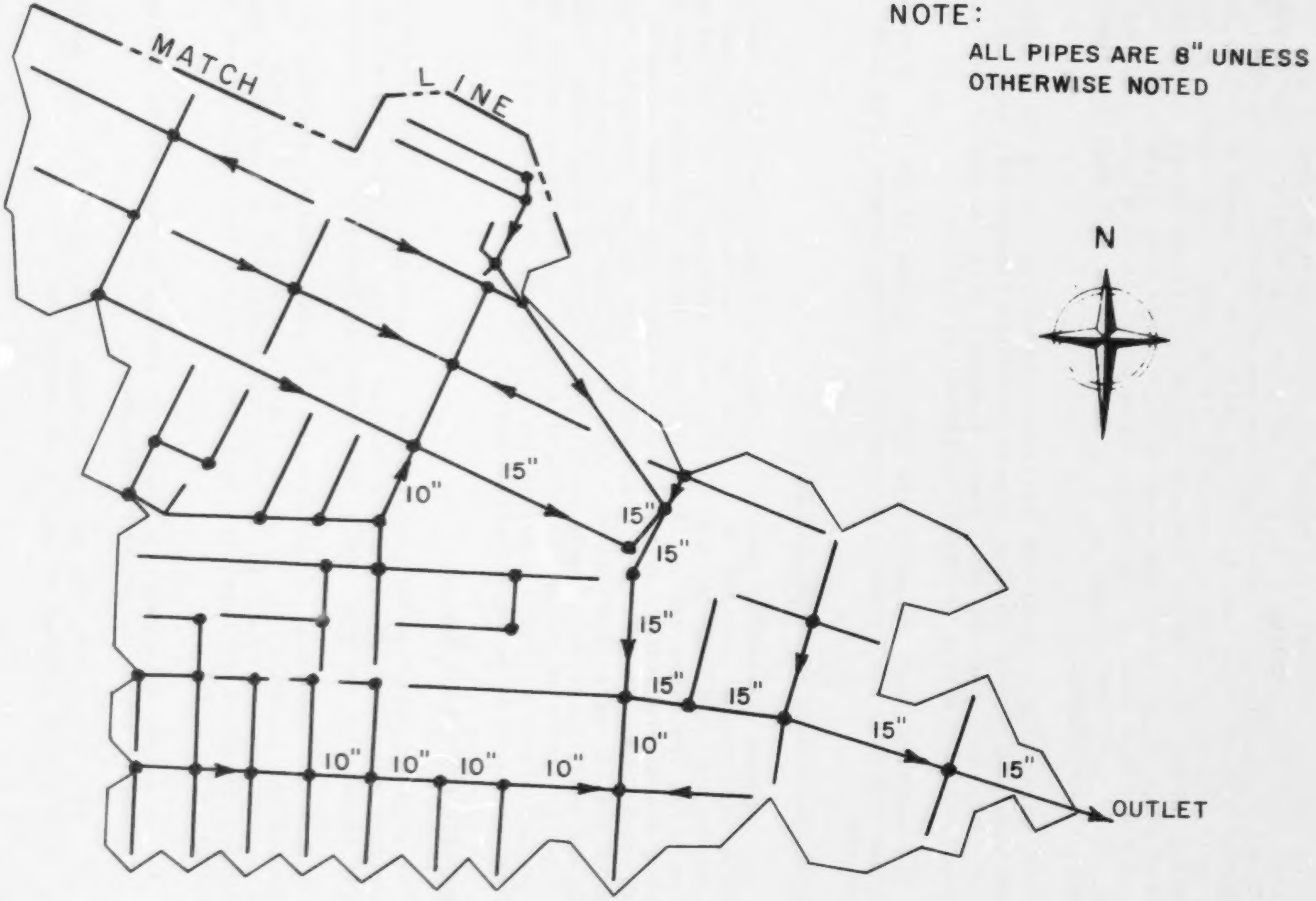

NOTE:
ALL PIPES ARE 8" UNLESS OTHERWISE NOTED
MATCH LINE
N
15"
10"
15"
15"
15"
15"
15"
15"
15"
10"
10"
10"
10"
10"
OUTLET

Figure 14. Sanitary sewer layout, subareas III, IV and V

The formula indicates that relatively higher maximum rates of flow are expected from small numbers of people than from large numbers. This formula was found applicable to Elizabeth in previous studies(8). For District A, the ratio of peak daily flow to average daily flow is computed as 2.74 and peak daily flow, as 11.82 m^3 per min. (4.50 mgd). For commercial and industrial areas, peak sewage flow was assumed as 187,000 liters per hectare (20,000 gallons per acre) and equaled 2.76 m^3 per min. (1.05 mgd). The peak dry-weather flow at the outlets was about 14.58 m^3 per min. (5.5 mgd). This peak was rounded up to 15.8 m^3 per min. (6.0 mgd) for computations in this report.

The minimum size used for sanitary sewers was 0.20 m (8 inches). The minimum pipe slopes selected assured a minimum flowing full velocity of 0.61 m (2.0 feet) per second. Sewer diameters were calculated based on Manning's equation with a roughness coefficient of 0.013.

The sanitary sewer diameters are shown in Figures 13 and 14. Most of the pipes are 0.20 m (8 inches) in diameter with the largest being 0.381 m (15 inches).

COMBINED SYSTEM WITH IN-SYSTEM STORAGE

As discussed previously, in-system storage, as considered in this study, includes (a) storage provided in enlarged upstream lateral sewers due to the economy achieved, and (b) satellite storage provided in the collection system along the trunk sewer. Both storage schemes have the effect of reducing the size of trunk sewers. In some cases, trunk sewers designed to convey peak flows are the practical maximum that can be physically constructed due to space limitations. Pipe storage provided in upstream laterals and satellite storage would permit a reduction in trunk sewer sizes. Such storage serves to reduce the peak of outflow hydrograph and modify the outflow pollutograph from a drainage basin thus reducing the capacity of interceptors required to abate overflow pollution.

Design with In-Pipe Storage

In-pipe storage can be provided in both lateral and trunk sewers. Preliminary cost analysis indicates storage in the laterals, rather than in the trunks, to be less costly for either pollution or flood control.

Figure 15 shows the incremental cost of pipe storage resulting from increasing the diameters of the pipes required to convey storm runoff. The pipe cost is obtained from Figure D-15 or TABLE D-17 for an excavation depth allowing 1.52 meters (5 feet) of cover over sewers. The following can be observed:

1. A slight enlargement of sewers to provide storage volume is less costly than a large sewer enlargement,

2. Pipe storage in the smaller lateral sewers would be less costly than pipe storage in large trunk sewers, and

3. Satellite storage which also reduces trunk sewer sizes may be the least costly alternative since in-pipe storage costs average about $0.26 per liter ($1.00 per gallon).

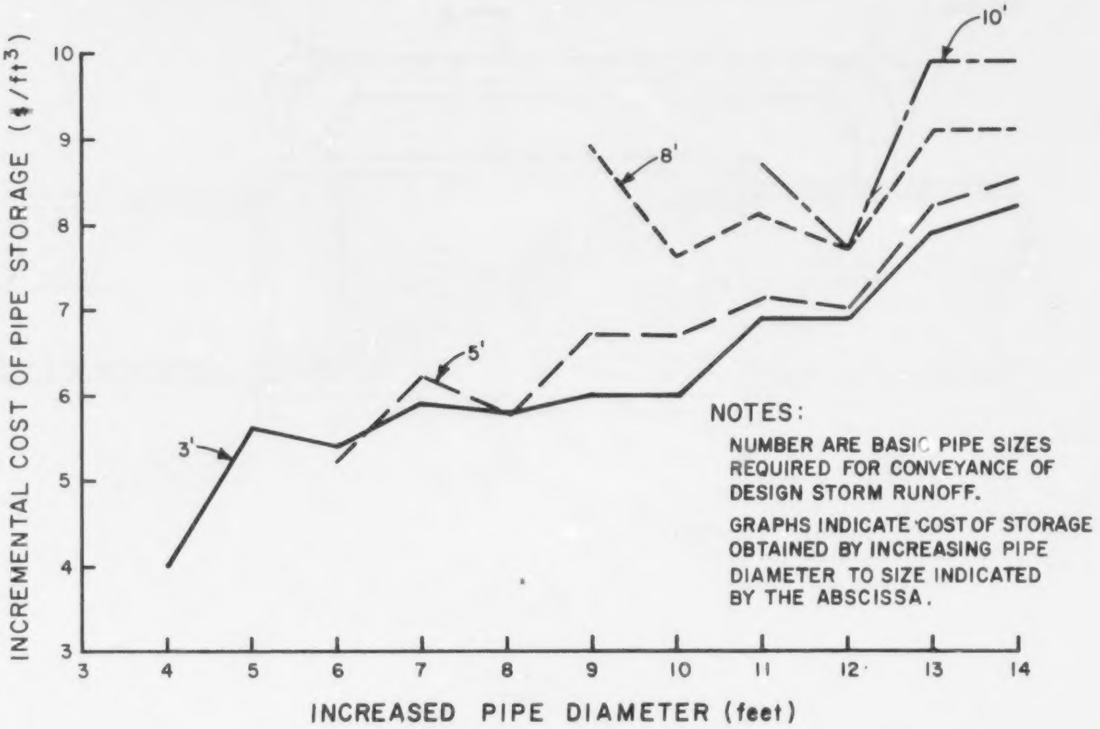


Figure 15. Incremental cost of pipe storage

In-pipe storage reduces peak runoff rates while permitting flow conveyance. The pipe volume, in excess of that required to pass peak flow, can provide regulation by installing either a downstream orifice coupled with an overflow weir (Figure 16) or a collapsible dam coupled with a regulator. Other devices such as the "Hydro-Brake"(13) and the VBB Flow Regulator (14) have been proposed. The backwater created by the regulators results in storing flows in excess of the desired downstream rate reaching the regulator. This reduces the design flow rate in the downstream sewers. It may also be designed to moderate the initial pollutant concentration by diluting the concentrated pollutant in the first flush with subsequent, less concentrated runoff.

The in-pipe storage analysis was accomplished in three steps.

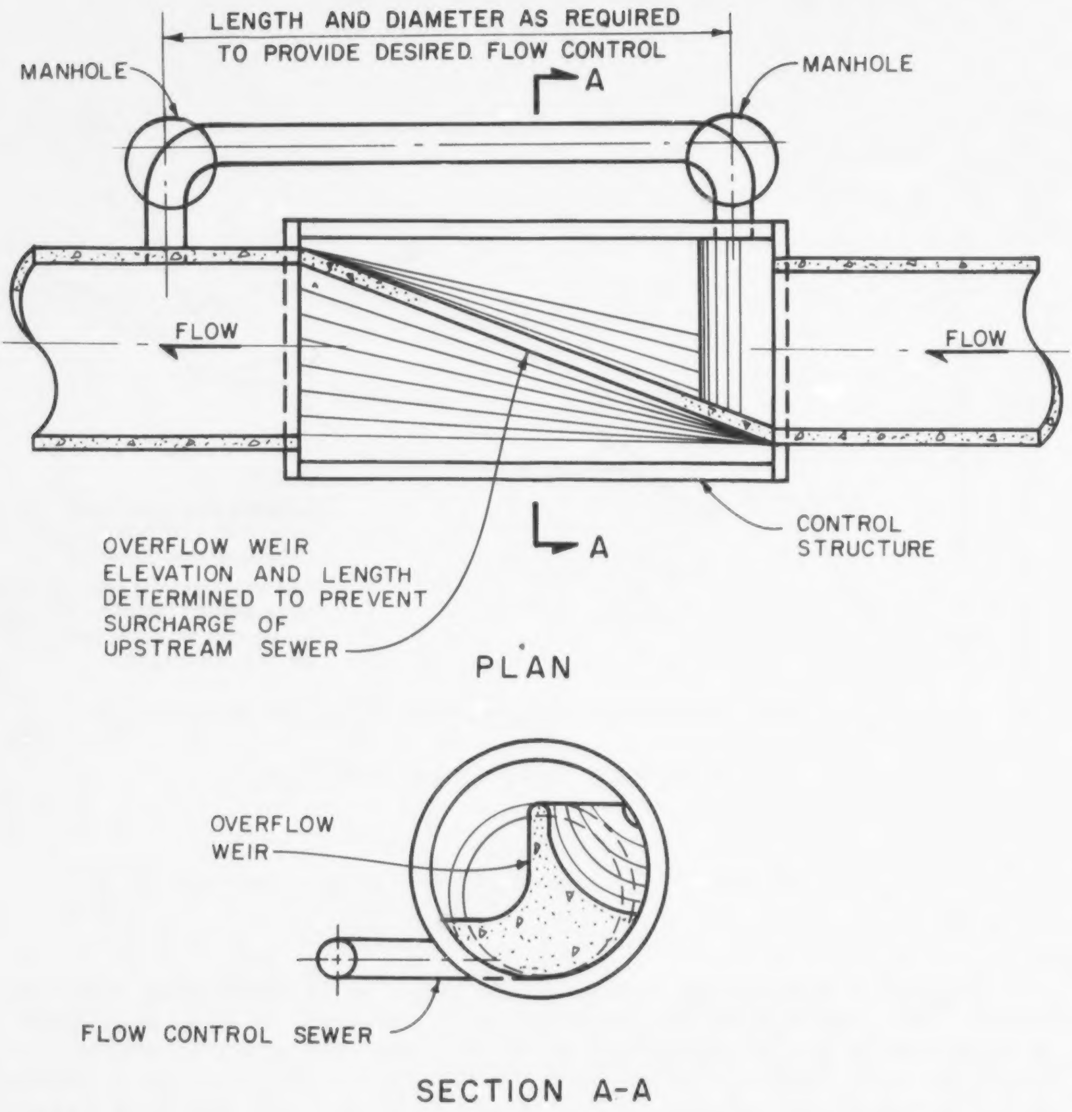

LENGTH AND DIAMETER AS REQUIRED
TO PROVIDE DESIRED FLOW CONTROL
MANHOLE
MANHOLE
A
FLOW
FLOW
A
CONTROL
STRUCTURE
OVERFLOW WEIR
ELEVATION AND LENGTH
DETERMINED TO PREVENT
SURCHARGE OF
UPSTREAM SEWER
PLAN
OVERFLOW
WEIR
FLOW CONTROL SEWER
SECTION A-A

Figure 16. Weir-orifice regulator

Step I - The overland flow hydrograph to lateral sewers from each subcatchment was generated for the 5-year synthetic storm using a computer program adapted from the SWMM RUNOFF Block. Summing arithmetically the corresponding time steps of the tributary subcatchment hydrographs, the overland flow hydrograph to a pipe element was derived. This hydrograph was integrated to obtain the total direct runoff. The volume that should be stored to obtain the desired outflow peak rate was then determined by integration of the flow in the overland flow hydrograph in excess of that rate. The peak constant flow rate shown in the modified hydrographs of Figure 17 assumes a variable orifice outlet control.

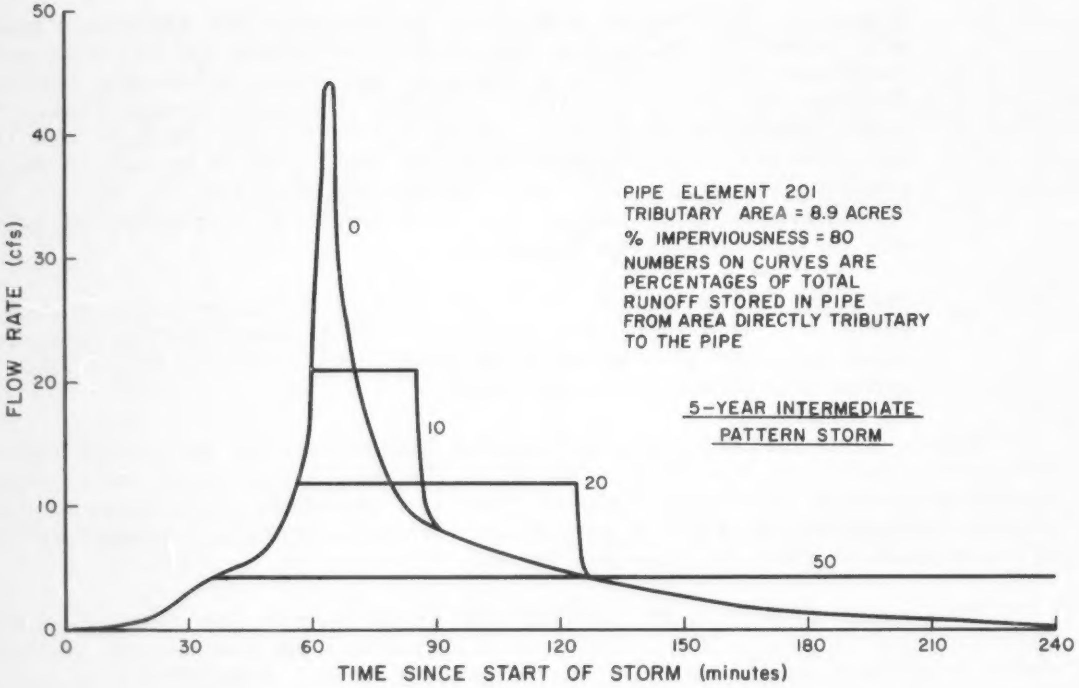
Step II - The desired peak flows of the upstream tributary sewers were summed to obtain the peak flow to be passed for the pipe under consideration. Manning's equation was used to compute the pipe diameter and slope required to convey the peak design flow. The pipe diameter required for storage was separately computed and then was combined with the pipe diameter for flow conveyance to obtain an aggregate design pipe size having the same cross-sectional area as the two separate pipes. The next available commercial pipe size was used as the design diameter.

Step III - The design of trunk sewers was made using the SWMM TRANSPORT Block with the modified hydrographs entering the trunk sewer from the lateral sewer branches. No surcharge was allowed in either the lateral or trunk sewers.

Figure 17 illustrates the controlled hydrographs to be routed through the pipe element 201 for storage of 0,10, 20 and 50 percent of total runoff volume drained to that pipe element from its tributary subcatchment. Zero percent storage represents the case without in-pipe storage. Storage reduces the magnitude of the peak flow but extends its duration.

The design diameters of lateral and trunk sewers for in-pipe storage volumes equal to 8.3, 16.5, 25.0 and 41.3 percent of the 5-year synthetic storm runoff volume in District A are listed in TABLE 1 and TABLE 2, respectively. These percentages correspond to a total of 10,200; 20,100; 30,300; and 50,300 m^3 (2.7, 5.3, 8.0 and 13.3 million gallons), respectively, stored in the lateral sewer pipes. The respective percentages of tributary subcatchment runoff volume stored in each lateral sewer element are, however, 10, 20, 30 and 50 since about 14 percent of District A drains directly to trunk sewers.

As the volume stored in lateral sewers increases, the lateral sewer sizes are consistently increased except for some at the lower end of lateral sewer branches. Trunk sewer sizes are consistently reduced because of the reduction in peak flows discharged by the branches. The reduction in size is reasonably uniform over the entire length of trunk sewer. This type of design provides storage for pollution control at least cost, as previously described.



PIPE ELEMENT 201
TRIBUTARY AREA = 8.9 ACRES
% IMPERVIOUSNESS = 80
NUMBERS ON CURVES ARE PERCENTAGES OF TOTAL RUNOFF STORED IN PIPE FROM AREA DIRECTLY TRIBUTARY TO THE PIPE

5-YEAR INTERMEDIATE PATTERN STORM

Figure 17. Hydrographs modified by in-pipe storage

Outflow Hydrographs and Pollutographs

Figure 18 presents outflow hydrographs for various amounts of runoff temporarily stored in upstream laterals. The peak runoff rates are 293, 228, 162, 126 and 93 m^3/sec (1,407, 1,096, 788, 607, and 448 cfs) for amounts of storage equal to 0, 8.3, 16.5, 25.0, and 41.3 percent respectively of about 119,000 m^3 (31.5 million gallons). The computer program developed in this project for in-pipe storage does not currently permit routing pollutants. Therefore, pollutographs were not generated.

The pollution abatement benefits of in-pipe storage depend on the flow rate selected at which storage of combined waste begins. Where a pollutant sink does not exist, the higher the flow-through rate, the smaller would be the benefit achieved and the smaller the amount of storage required. Less pollution abatement benefit results since less dilution of the high pollutant concentration in the first flush would occur.

Increasing the amount of in-pipe storage reduces the flow velocity and may result in solids settlement in the sewers. About 60 percent of the pipes, when in-pipe storage equals 8.3 percent of the total 5-year synthetic storm runoff volume, would have a flowing full velocity greater than 0.30 m (1.0 feet) per second. As the amount of storage increases to 25 percent, less than 20 percent have flowing full velocity exceeding 0.30 m (1.0 feet) per second with about 40 percent have a velocity less than 0.06 m (0.2 feet) per second. Solids deposited in sewer pipes during the pipe full, low velocity period may be resuspended as flow recedes and velocity increases. However, where in-pipe storage of about 40% of the overland flow volume is provided, additional maintenance for cleaning may be required. The Boston sewer flushing study (15) indicates that sewage impounded in upstream sewer segments by manhole stoppers are self-cleansing upon stopper release due to sudden increase in velocity with pipe drawdown.

In a wastewater facilities plan study for Trenton, New Jersey (16), several wet-weather flows were sampled to find the values of BOD and SS in combined sewage. Automatic samplers were used and started at the beginning of the rainfall with samples taken at 15-minute intervals. The results of two sampling periods during small storms are shown in Figure 19. An increase in BOD occurred in a short time as the storm runoff increased flow to the treatment plant. The BOD then receded as the sewage flow exceeded the treatment plant's pumping capacity and sedimentation occurred in the interceptor due to reduced velocities resulting from storage in the sewer. After the end of the rainfall, wastes stored in the interceptor were drawn upon, the sewer velocity increased, settled material was resuspended and peak concentrations of BOD reached the plant.

The deposition and resuspension of solids would be reflected in the outflow pollutographs.

SEWERS WITH SATELLITE STORAGE

Satellite storage along the trunk was investigated using the SWMM TRANSPORT Block. The simulation included no more than two such basins. The

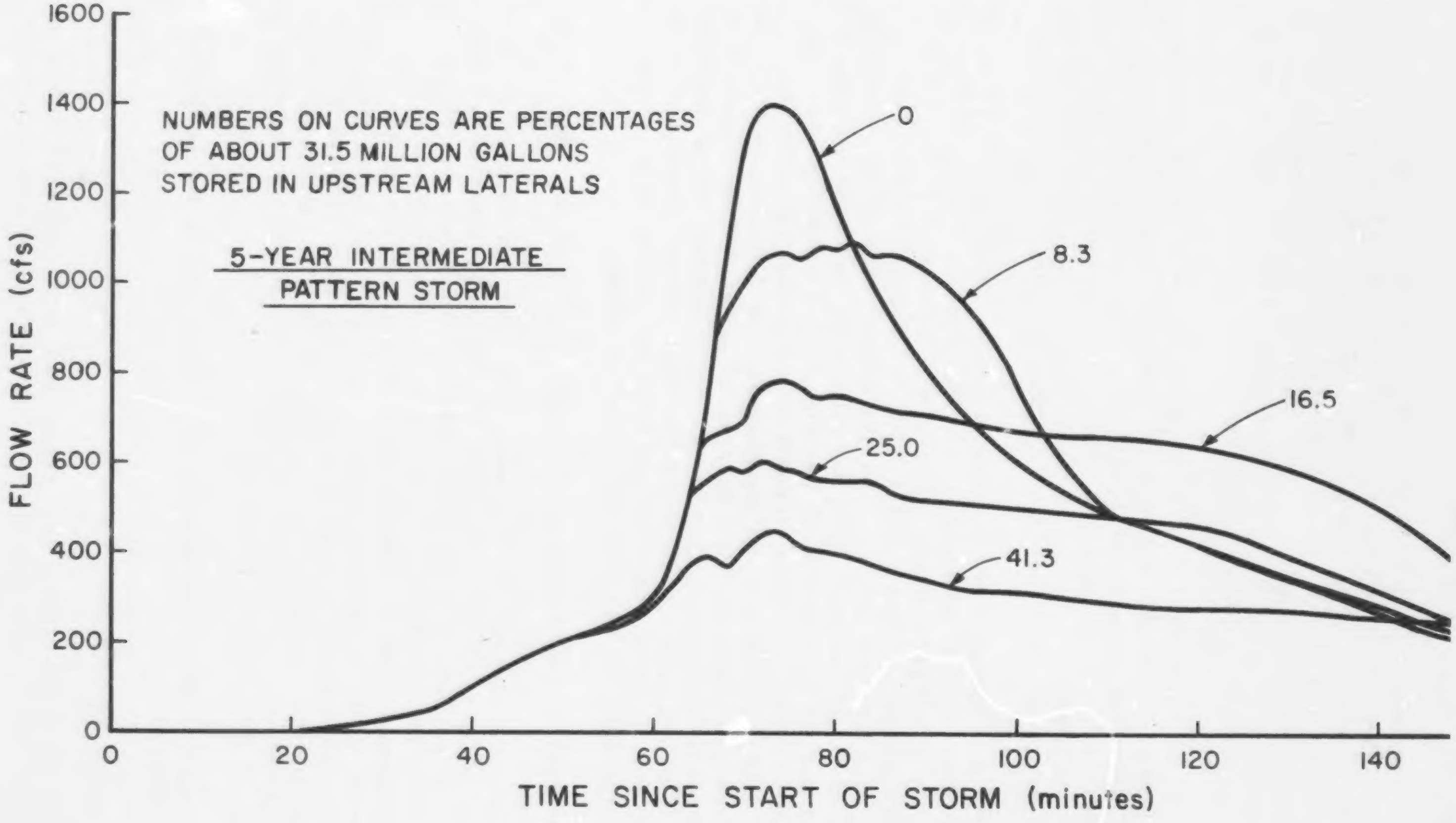


Figure 18. Outflow hydrographs with in-pipe storage (5-year intermediate pattern storm)

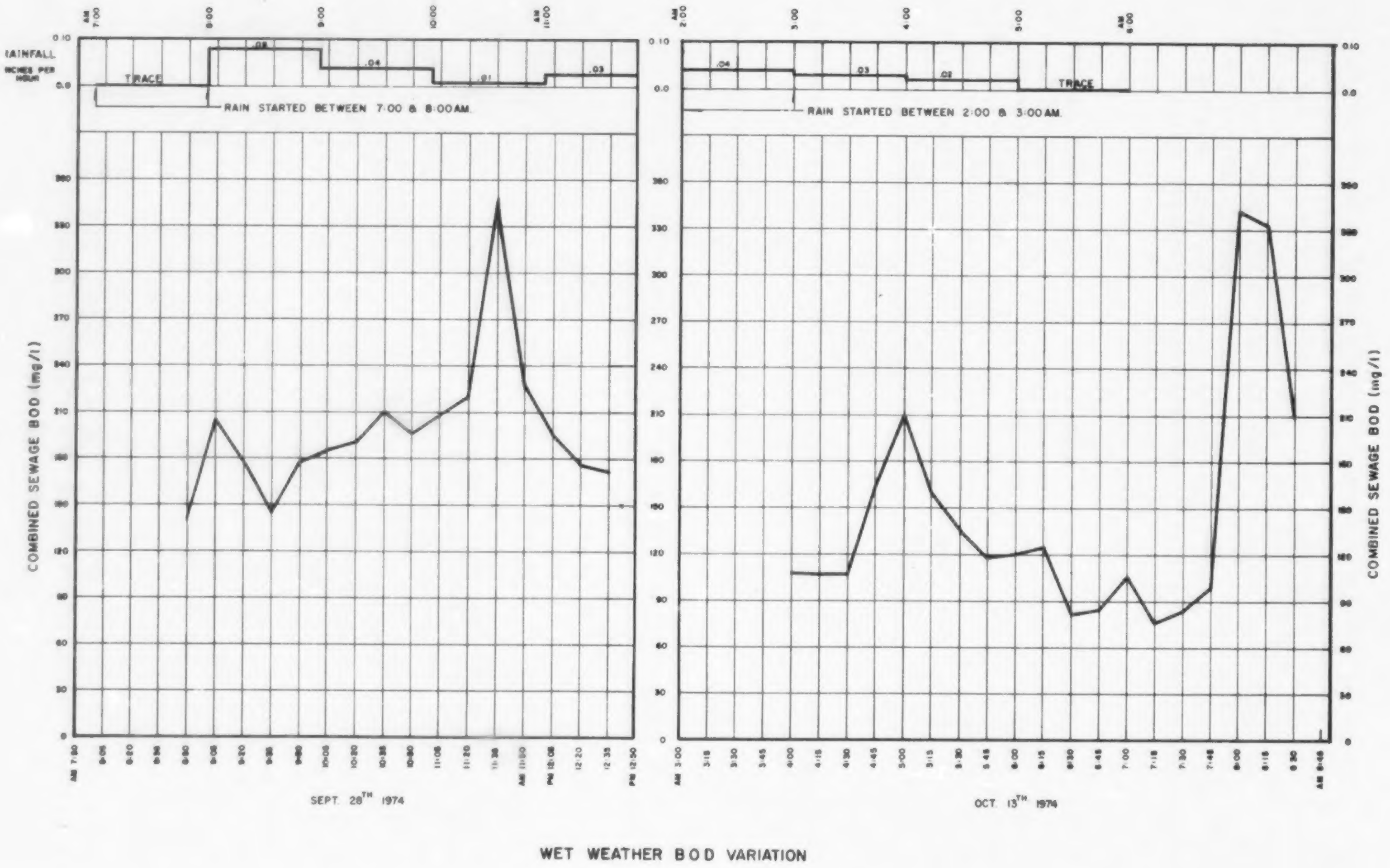


Figure 19. Wet-weather BOD variation, Trenton, New Jersey

effect of storage quantity and location on the trunk sewer sizes and outflow hydrographs and pollutographs was studied. All flows were assumed diverted to storage basins with outflow occurring only after storage was full. In addition, at any point in time, the flow contained in a storage basin was assumed to be completely mixed. After storage basin locations were determined for various amounts of storage capacity, runoff characteristics for various combinations of storm intensity and patterns were developed. These runoff characteristics were used in determining comparative costs.

Design

Design of trunk sewers with satellite storage was made with SWMM. Cylindrical storage basins were assumed with outlets controlled by overflow weirs. The width of the overflow weir was determined to minimize the hydraulic head above the crest of the weir and reduce the effect of surcharge volume on the routing of storm runoff. Since the flow divider option in SWMM was not used, dry-weather flows above a storage basin are intercepted by the basin. Consequently, the amount of solids deposit in the sewer reflected the location of storage basins. In practice, dry-weather flows would be diverted from the basin.

The locations and volumes of storage basins in a drainage watershed were selected to conform to the following criteria:

1. The peak of outflow hydrograph should be attenuated as much as practical.

2. For outflow hydrographs with multiple peaks, the peaks should be relatively equal to reduce intercepting sewer sizes.

3. The rising limb of the outflow hydrograph should be delayed as much as possible to provide reduction of the high concentrations of pollutants in the initial flush by dilution with later runoff.

Figure 20 shows the outflow hydrographs for storage basins at six different locations in District A. In each case, about 50,300 m^3 (13.3 million gallons) of the total 119,000 m^3 (31.5 million gallons) of assumed storm runoff is captured. TABLE 8 compares the corresponding trunk sewer sizes. The curve labeled "no storage" applies to a conventional combined system and is shown for reference only. Figure 21 presents schematically the various alternatives for storage location.

Case A assumes a single basin located at the downstream end of the system. Overflow occurs 82 minutes after the start of rainfall assuming the 5-year synthetic storm hyetograph. The peak overflow rate is 25 m^3/sec (890 cfs). The trunk sewer diameters are the same as in the conventional combined system, and the cost is about 75 percent more than Case B, the least expensive.

In Case B, the two storage basins of equal capacity were located at points where 26 and 54 percent of the watershed drained to them. The peak runoff is about the same as Case A but the peak occurs about 20 minutes

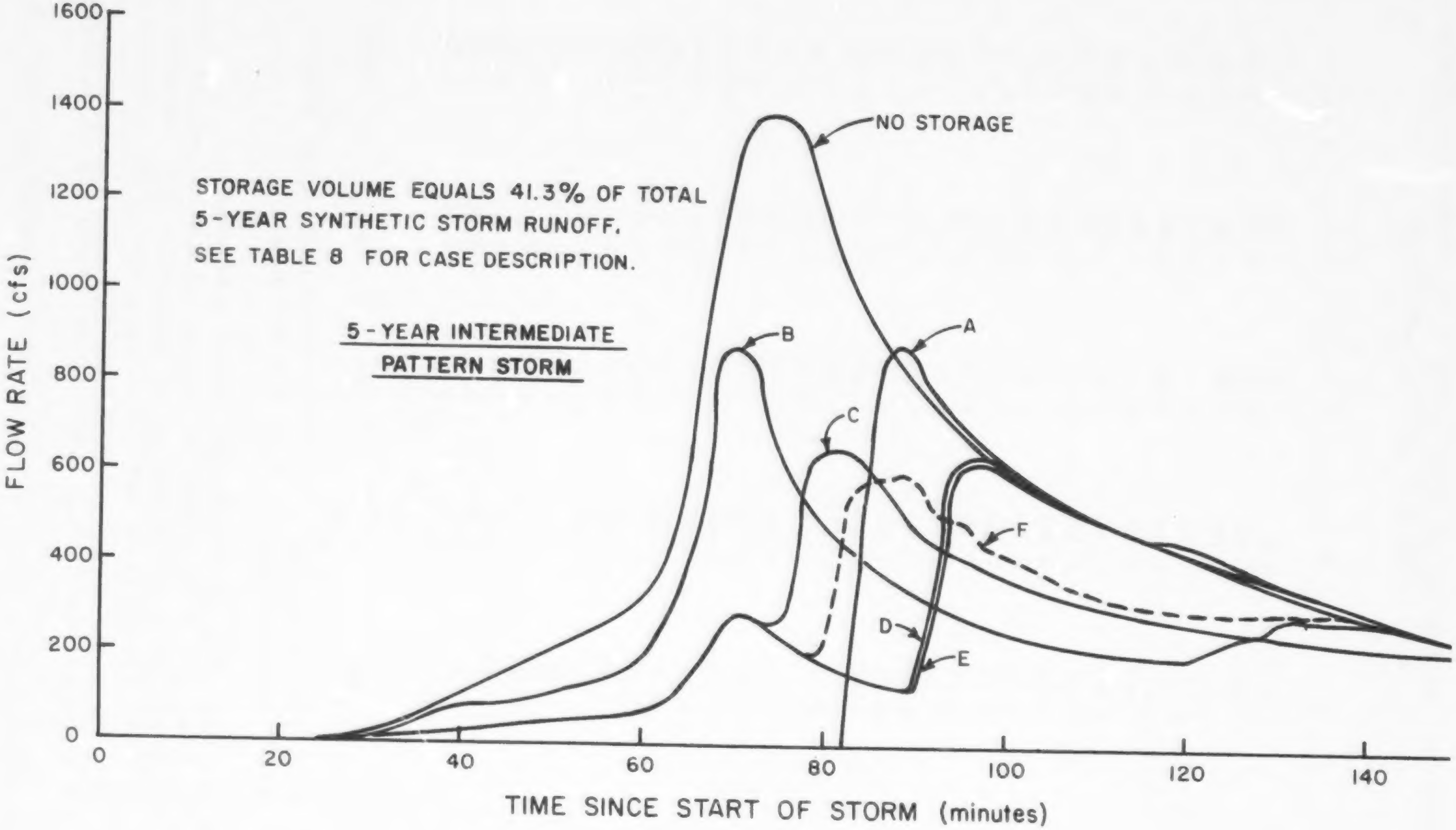


Figure 20. Outflow hydrographs with satellite storage at various locations (5-year intermediate pattern storm)

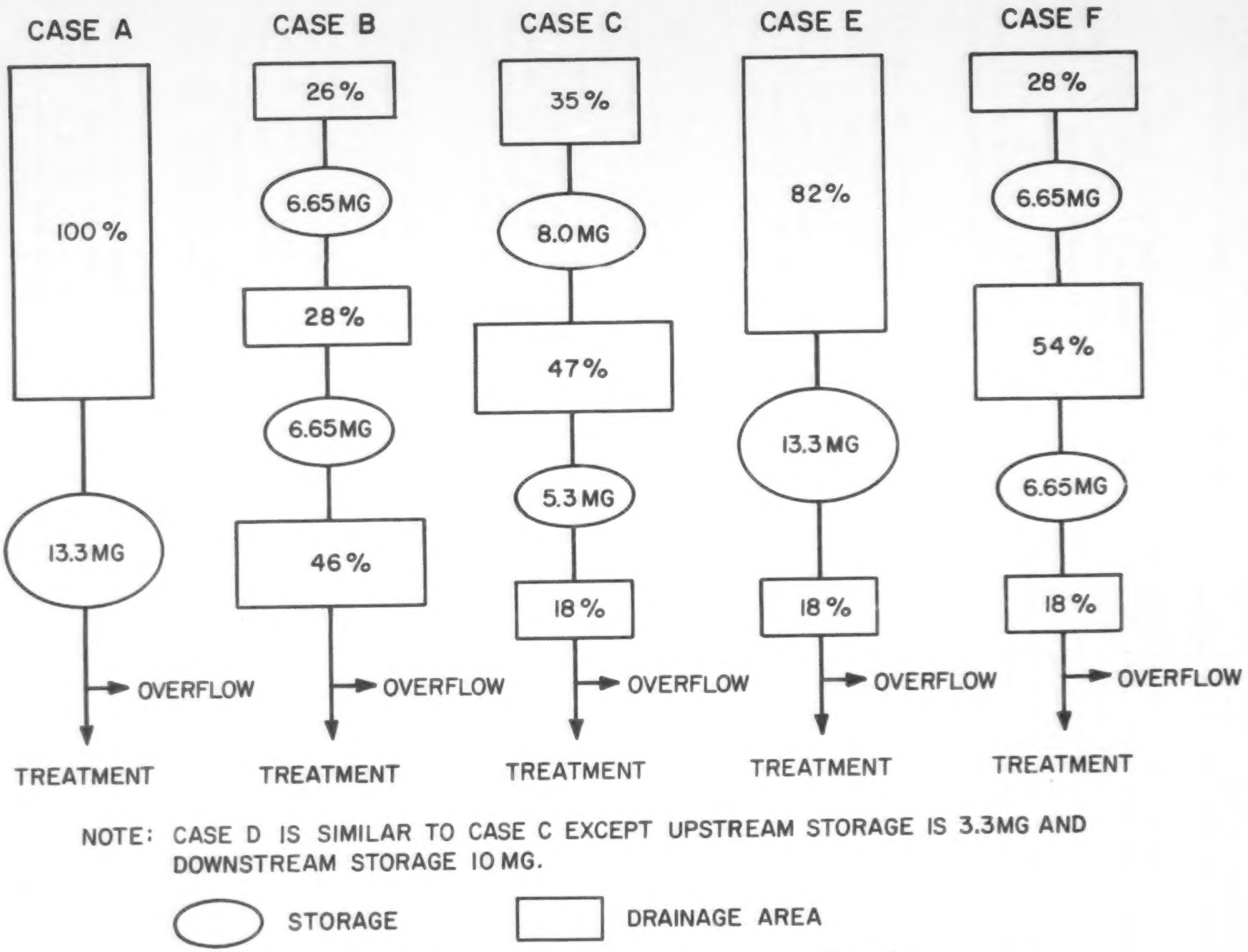
TABLE 8. COMPARISON OF TRUNK SEWER SIZES (FEET) FOR STORAGE BASIN AT VARIOUS LOCATIONS

Sewer Element No	A	B	C	D	E	F
301	3.0	3.0	2.75	2.75	2.75	2.75
303	3.50	4.00	3.50	3.50	3.50	3.50
305	4.00	4.00	4.00	4.00	4.00	4.00
307	4.50	4.50	4.50	4.50	4.50	4.50
309	5.00	5.00	5.00	5.00	5.00	5.00
311	6.00	6.00	5.50	5.50	5.50	5.50
313	6.00	6.50	6.00	6.00	6.00	6.00
315	6.50	7.00	6.50	6.50	6.50	6.50
317	7.00	4.00	7.50	7.50	7.50	7.50
319	6.00	3.50	6.00	6.00	6.00	6.00
321	8.00	4.50	8.00	8.00	8.00	5.00
323	8.00	4.50	8.00	8.00	8.00	4.50
325	9.00	6.00	5.00	9.50	8.00	5.00
327	6.50	5.00	4.00	7.00	6.50	4.00
329	8.00	5.50	4.50	7.50	7.50	5.00
331	9.00	7.00	5.00	8.50	8.50	6.00
333	6.50	5.00	4.00	6.00	6.50	4.50
335	7.50	6.00	4.50	7.00	7.00	4.50
337	7.50	6.00	4.50	7.00	7.00	5.50
339	9.50	7.50	6.50	8.50	9.00	7.00
341	10.00	5.00	7.00	9.00	10.00	8.00
343	10.00	5.00	7.00	9.00	10.00	8.00
345	10.00	5.00	7.00	9.00	10.00	8.00
347	13.50	6.50	9.50	12.00	13.00	10.00
349	13.00	7.00	9.00	12.00	13.00	11.00
351	14.00	7.00	10.50	12.00	13.50	10.50
353	12.50	9.00	9.50	9.50	9.50	9.50
357	11.00	8.50	8.50	8.50	8.50	8.00
359	11.00	8.50	8.00	8.50	8.00	8.00
361	11.50	9.00	8.50	8.00	8.00	8.00
363	11.50	9.50	8.50	8.50	8.50	8.00
Peak Runoff Rate (cfs)	890	897	665	644	630	607

TOTAL STORAGE VOLUME = 13.3 MILLION GALLONS OR 41.3% OF TOTAL 5-YEAR SYNTHETIC STORM RUNOFF VOLUME

NOTE: Case A - One basin at downstream end of watershed
B - Two basins of equal capacity at 26 and 54% of watershed area
C - Two basins at 35 and 82% of watershed area, capacity ratio 3:2
D - Two basins at 35 and 82% of watershed area, capacity ratio 1:3
E - One basin at 82% of watershed area
F - Two basins of equal capacity at 28 and 82% of watershed area.

1 foot = 0.3048 meters; 1 cfs = 0.0283 m^3/sec; 1 million gallons = 3,785 m^3


CASE A
100 %
13.3 MG
OVERFLOW
TREATMENT
CASE B
26 %
6.65 MG
28 %
6.65 MG
46 %
OVERFLOW
TREATMENT
CASE C
35 %
8.0 MG
47 %
5.3 MG
18 %
OVERFLOW
TREATMENT
CASE E
82%
13.3 MG
18 %
OVERFLOW
TREATMENT
CASE F
28 %
6.65 MG
54 %
6.65 MG
18 %
OVERFLOW
TREATMENT
STORAGE
DRAINAGE AREA

NOTE: CASE D IS SIMILAR TO CASE C EXCEPT UPSTREAM STORAGE IS 3.3MG AND DOWNSTREAM STORAGE 10 MG.

Figure 21. Schematic diagram of satellite storage locations, District A

earlier. Hence, more polluted runoff would have to be intercepted and more off-line storage capacity would be required. On the other hand, trunk sewer sizes are substantially reduced.

Case C has a lower peak in the outflow hydrograph than Case B with about the same cost of trunk sewers. Two basins, having a capacity ratio of 3:2, were located where 35 and 82 percent of the watershed drains to them. Further improvements of the outflow hydrograph is achieved by using the same basin locations but with a capacity ratio of 1:3 (Case D). However, trunk sewer sizes are generally greater than those in Case C. Case E, with one large basin located where 82 percent of the area drains to it, has a comparable outflow hydrograph and trunk sewer sizes (except for the reaches immediately upstream of the basin) as those of Case D.

Case F assumes two basins of equal capacity located where 28 and 82 percent of the watershed drains to them (trunk sewer element numbers 320 and 352). The outflow hydrograph improved somewhat over that of Case C with trunk sewer sizes generally comparable. Based on the criteria stated previously for the selection of basin location, Case F offers most advantage and was used to represent the desirable locations for storage equalling 41.3 percent of the 5-year synthetic storm runoff. In practice, the locations for storage would be dictated by site conditions.

For satellite storage volumes equal to 9.4 and 17 percent of 5-year storm runoff, two basins of equal capacity located where 26 and 54 percent of watershed drains to them, or at trunk sewer element numbers 316 and 340 respectively, appear to result in reasonably good outflow hydrographs (Figure 22). The two peaks in the hydrographs are of about the same magnitude and the reduction in peak flow rate is appreciable. An additional computer run was made for 17 percent storage with basins located where 28 and 82 percent of the watershed drained to them. The peak of the outflow hydrograph was about 15 percent greater and the trunk sewer sizes slightly greater.

The trunk sewer diameters for three different amounts of satellite storage are listed in TABLE 2 together with the sewer sizes for a conventional combined system and advanced systems with in-pipe storage. The effect of storage on sewer sizes is more pronounced for capacities of about 17 or more percent of the total runoff. A major reduction in sewer sizes results as the storage capacity increases to 41.3 percent of the 5-year synthetic storm runoff volume. For a storage capacity equal to 9.4 percent of the total 5-year synthetic storm runoff volume, the trunk sewer diameters are generally less than that for conventional combined system but not by a significant amount. Similar findings have been reported by Yen and Sevuk(17).

The sewer size of element number 317 for 9.4 percent satellite storage is greater than for a conventional combined system. This is caused by the release of waste over a gravity weir after the storage basin is full. The rate of release is affected by the plan area of the basin and the characteristics of the overflow weir. In this particular case, the effect of basin

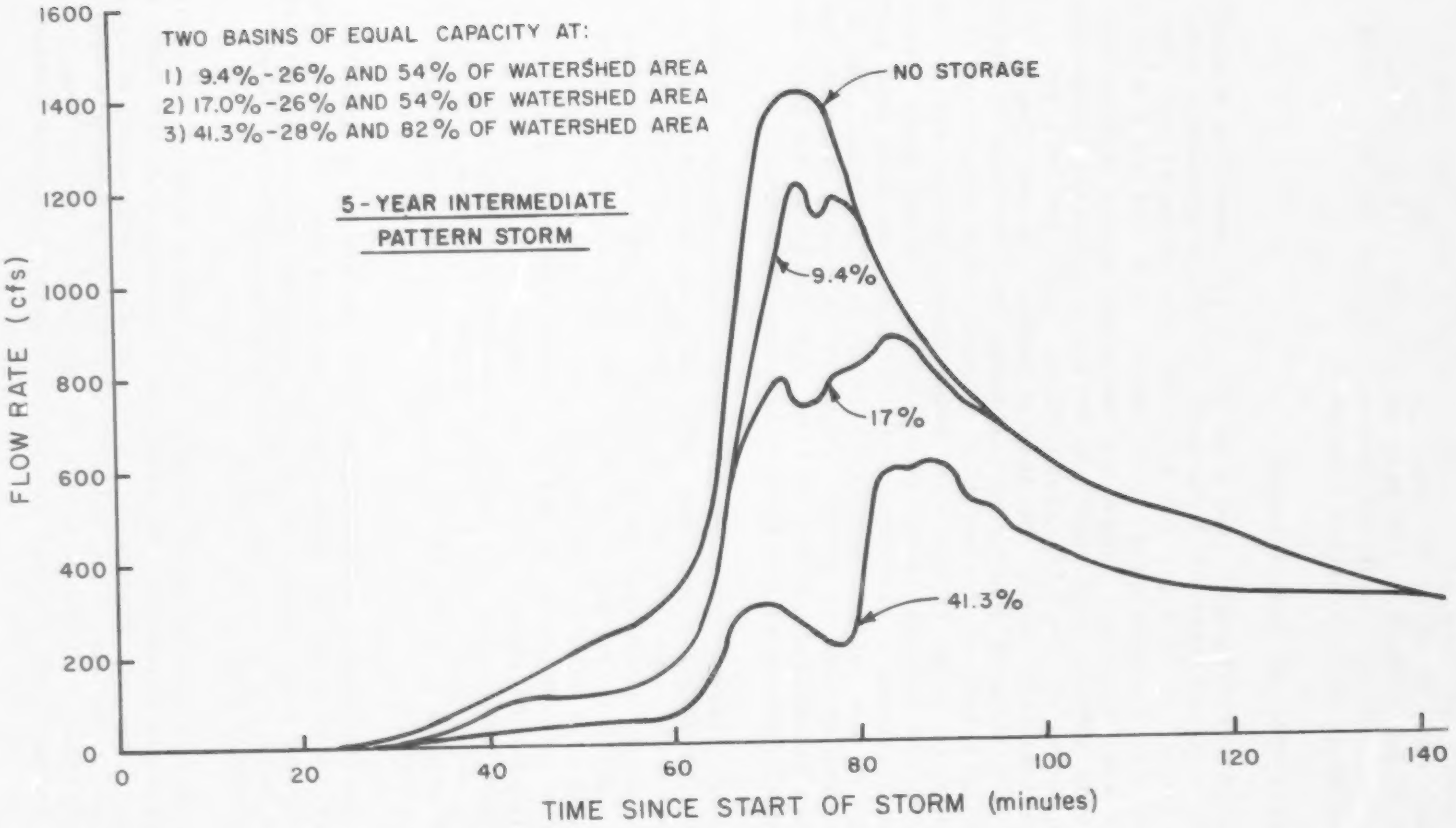


Figure 22. Outflow hydrographs with satellite storage (5-year intermediate pattern storm

volume on discharge rate is of particular significance since the storage basin becomes full at about the same time as the peak inflow, and hence, is not available for storage. The same explanation applies in the case of 17 percent storage to pipe elements numbers 321 and 323 whose diameters are greater than those of 9.4 percent storage.

Outflow Hydrographs and Pollutographs

The outflow hydrographs from District A, for satellite storage amounts equal to 0, 9.4, 17, and 41.3 percent of 5-year synthetic storm runoff (about 119,000 m^3 or 31.5 million gallons), are in Figure 22. The peaks of hydrographs are respectively 39.8, 34.2, 24.9 and 17.2 m^3/sec (1,407 ,1,208, 879 and 607 cfs). There are two peaks in the outflow hydrographs. The first peak results from runoff below the downstream storage basin. The first 60 minutes of the hydrograph rising limbs for 9.4 and 17 percent storage are identical, since the storage basins are identically located in each case. The shape of hydrograph following the first peak is influenced by the modulating effect of the unfilled storage volume remaining, if any, and the reduction in local runoff from areas downstream of the second basin. Overflow from the storage basin can result in a second peak. With greater storage (41.3 percent), the first peak is less than with less storage (17 percent) due to the further downstream location of storage. The first peak for 9.4 percent storage is greater than for 17 percent storage, due to the small storage volume and earlier spillage from basins. The time lag between the two peaks increases as the storage capacity increases.

Figure 23 presents the 5-year, intermediate-pattern, synthetic storm outflow SS concentration and mass rate pollutographs from District A for various amounts of satellite storage. The effect of satellite storage on pollutant loadings discharged is indicated. The peak concentration of SS is lower with satellite storage. Although the peak SS concentration for 41.3 percent storage is higher than for 9.4 and 17 percent storage, this is not significant with respect to pollution control since the flow rates are low when the high concentrations occur. For example, at the time of the peak SS concentrations for 0, 9.4, 17, and 41.3 percent storage, the flow rates are respectively 0.13, 0.07, 0.07 and 0.02 m^3/sec (4.6, 2.6, 2.6 and 0.6 cfs). At the time of peak outflow, the SS concentrations are respectively 7.9, 8.6, 5.8 and 13.4 mg/l.

Based on the current SWMM program, the low discharge pollutant concentrations of intense storms imply that intercepting of urban runoff for treatment during such times may not be required where the concentration, rather than mass, of pollutants discharged is governing and there is not a need for extremely high quality water.

The higher SS concentration with 41.3 percent storage is coincident with peak storage basin spillage. The same peak concentrations are found with 9.4 percent and 17 percent storage but at an earlier time since spillage takes place earlier. The mass rate pollutograph for 41.3 percent storage during the first 70 minutes of storm runoff is substantially less than for 9.4 and 17 percent storage. No storage results in highest pollutant levels. Hence,

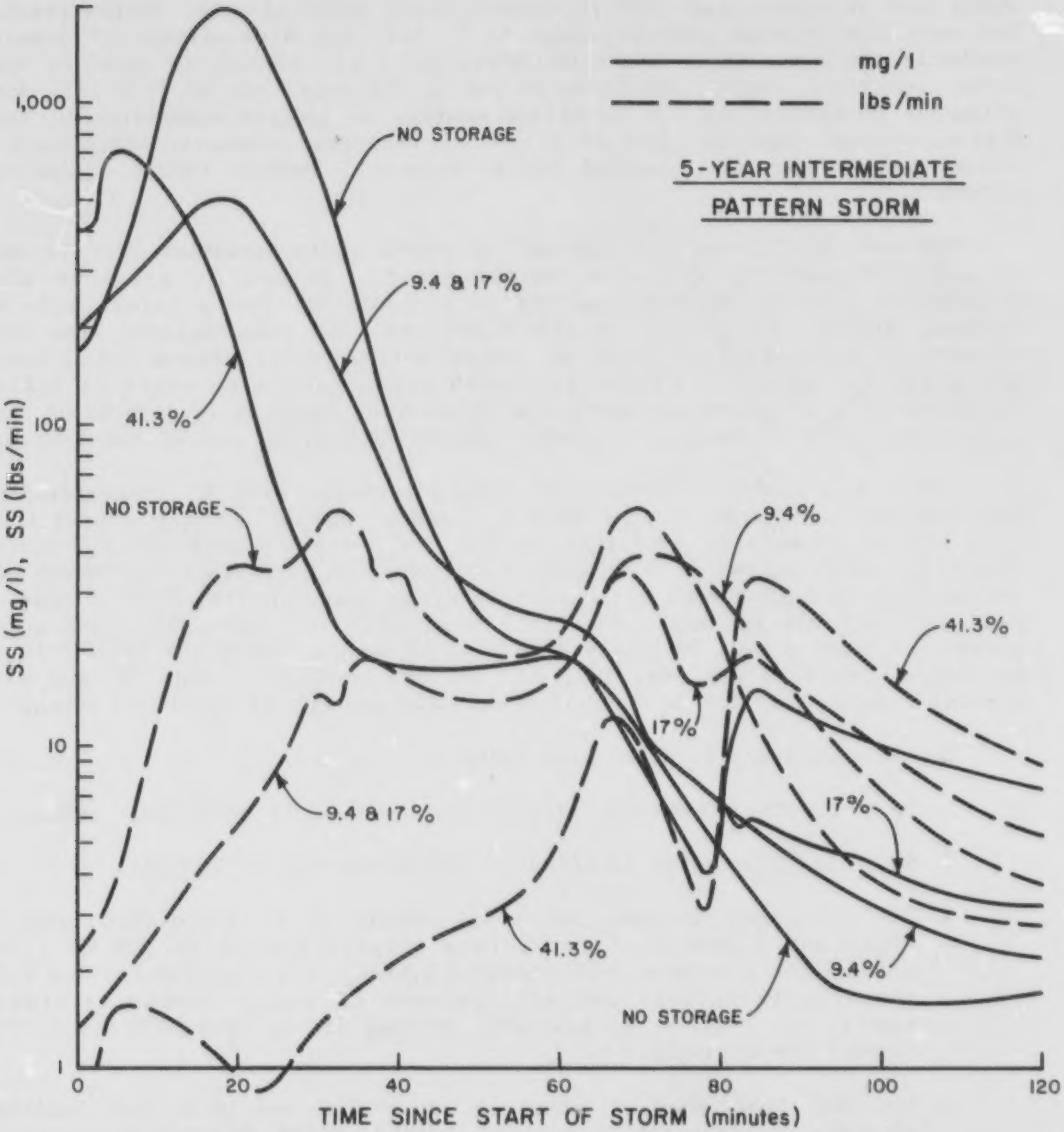


Figure 23. Outflow pollutographs with satellite storage (5-year intermediate pattern storm)

satellite storage provides benefit in controlling both storm runoff quantity and quality.

Figures 24 and 25 present the 1-year and 1.3-month intermediate pattern storm outflow hydrographs and SS concentration pollutographs, respectively, for satellite storage amounts equal to 0, 9.4, and 41.3 percent of 5-year synthetic storm runoff. Their characteristics are similar to those of the 5-year synthetic storm. Differences are in the magnitude of flow rate and pollutant concentration. If satellite storage is located downstream of the system, storage capacity equal to 17 percent of 5-year synthetic storm runoff volume may contain all combined sewage from a 1.3-month return frequency rainfall.

The effect of satellite storage on runoff characteristics from storms of advanced pattern was also investigated. Figure 26 presents the District A outflow hydrographs and pollutographs for a 1-year return interval storm. The shapes of the hydrographs and pollutographs from advanced pattern synthetic storms of 5-year and 1.3-month return frequencies are similar. Runoff from the 1.3-month synthetic storm would be fully contained by a satellite storage basin of capacity equal to 17 percent of the 5-year synthetic storm runoff located at the downstream end of the system.

TABLE 9 provides a summary of peak discharge, peak SS concentration and the total SS mass loading from a combined system serving District A with various amounts of satellite storage for 5-year, 1-year and 1.3-month return frequency storms of intermediate and advanced patterns. The amount of solids deposited in sewers during dry days, as computed with SWMM is dependent upon the storage basin locations selected. For the cases being compared, the total pounds of solids deposited in sewers during the assumed four antecedent dry days are 509, 172, 172 and 395 kg (1,124, 380, 380 and 872 pounds) respectively for 0, 9.4, 17.0 and 41.3 percent of satellite storage.

The following may be noted from TABLE 9:

1. Peak runoff rates are effectively reduced by satellite storage.

2. Satellite storage is an effective pollution control device.

3. The difference between the total pounds of SS discharged from 5-year and 1.3-month intermediate pattern storms is 328 kg (724 pounds) for a conventional combined system. For a system storing 9.4 percent, 17 percent and 41.3 percent of 5-year synthetic storm runoff, the differences are 429, 396 and 553 kg (948, 875 and 1,221 pounds) respectively.

4. The peak flow, peak pollutant concentration, and total mass loading discharge is less for an advanced pattern storm than for an intermediate pattern storm of the same frequency.

Satellite storage basins along the trunk line of a combined system result in the reduction of trunk sewer sizes to control both flooding and

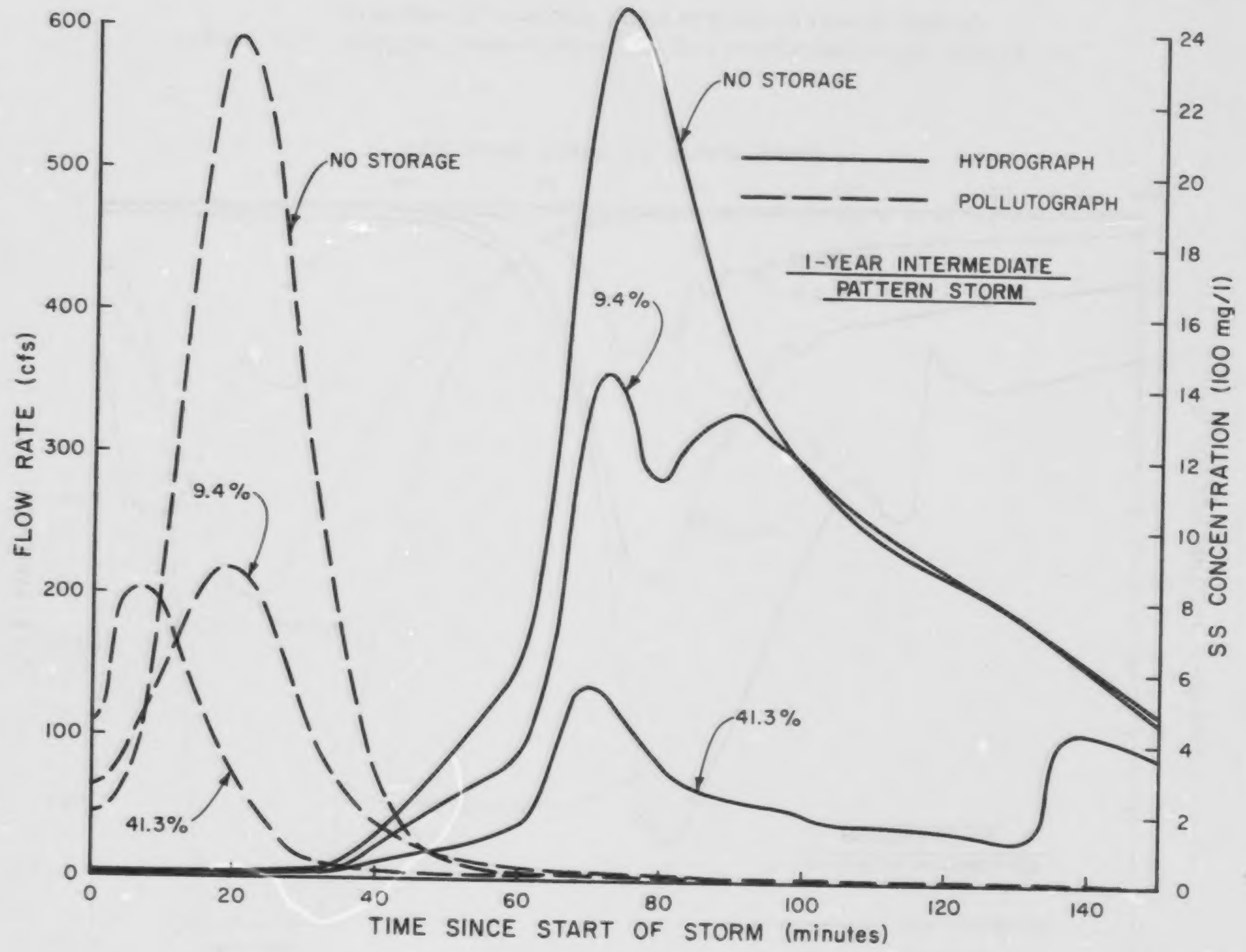


Figure 24. Outflow hydrographs and pollutographs with satellite storage (1-year intermediate pattern storm)

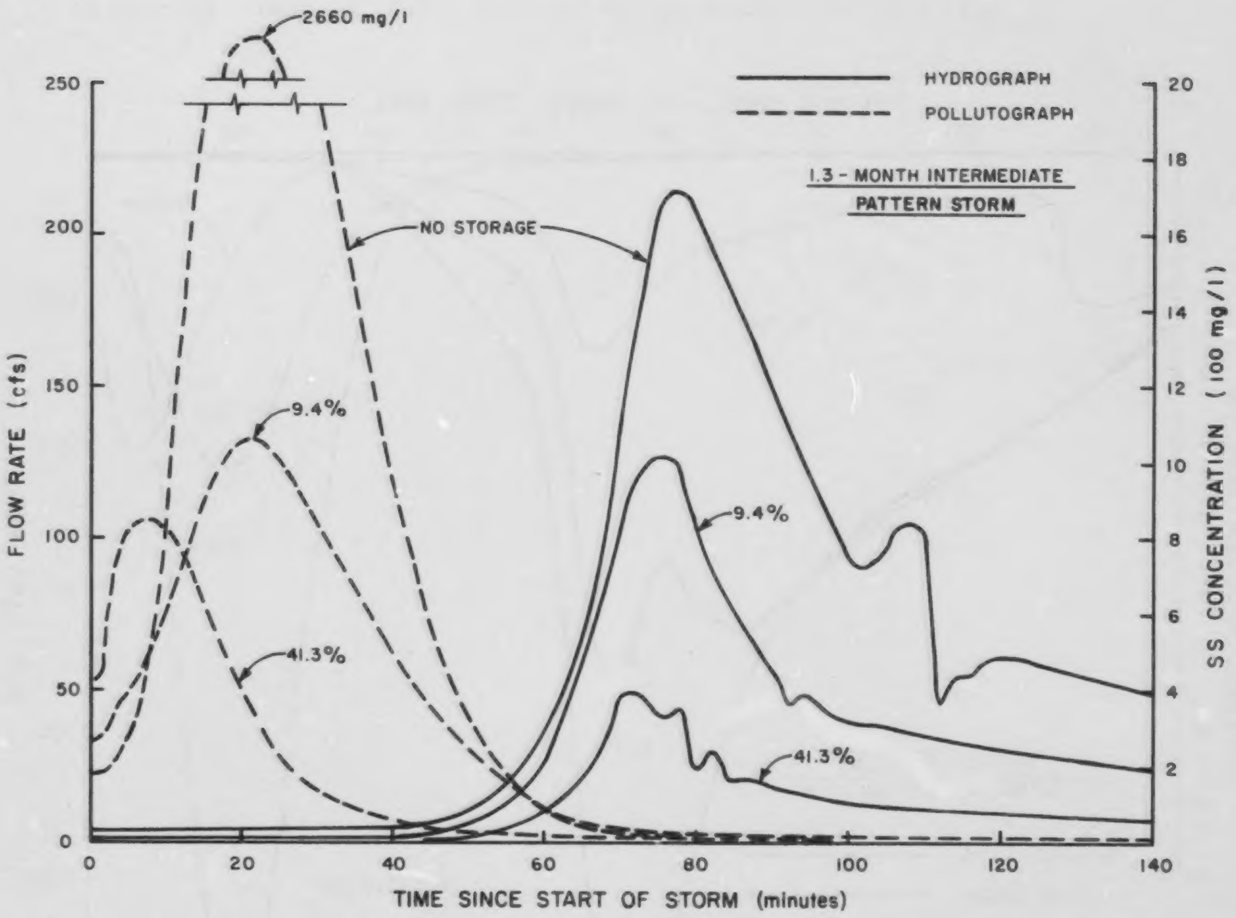


Figure 25. Outflow hydrographs and pollutographs with satellite storage (1.3-month intermediate pattern storm)

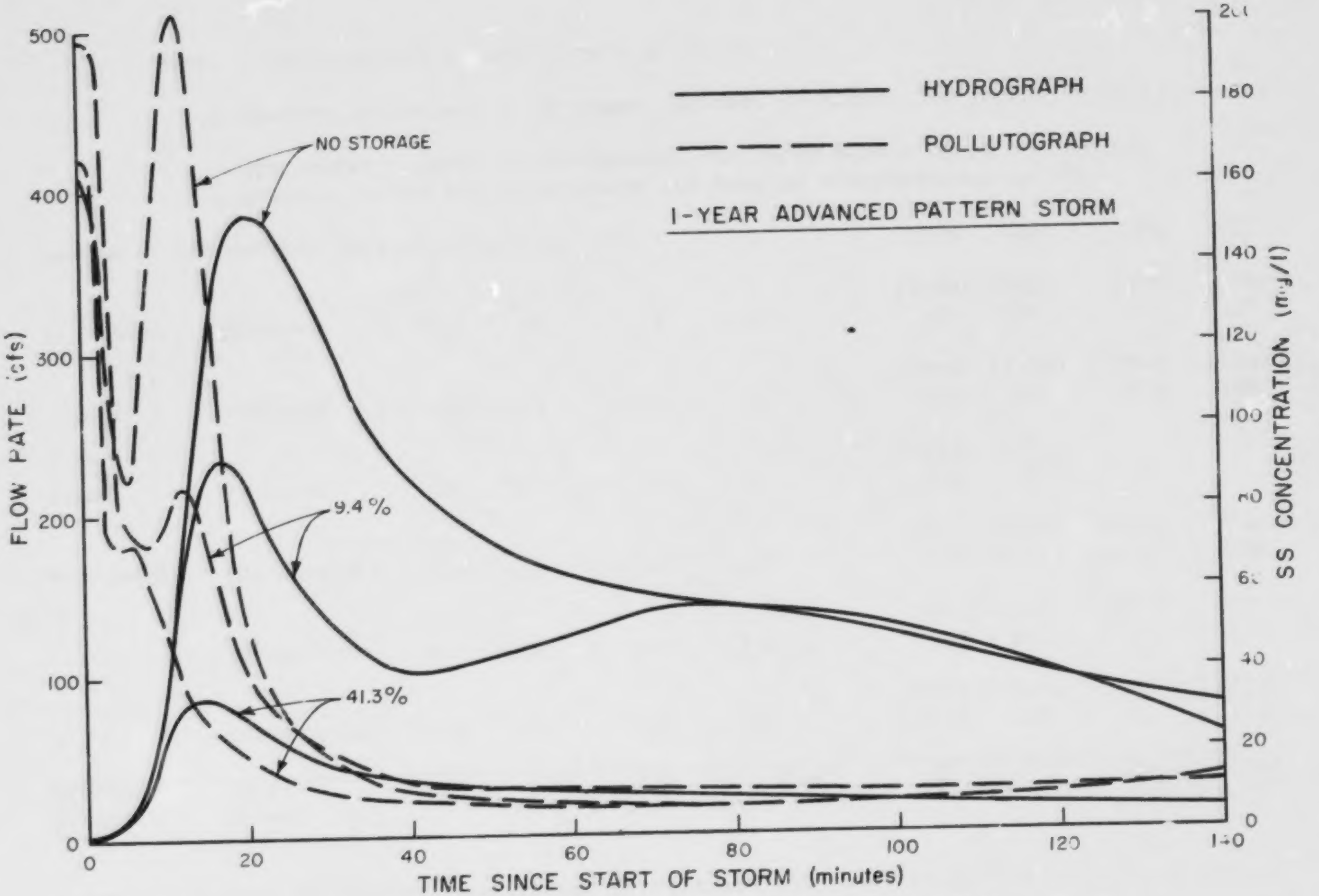


Figure 26. Outflow hydrographs and pollutographs with satellite storage (1-year advanced pattern storm)

TABLE 9. SUMMARY OF RUNOFF QUANTITY AND QUALITY FOR COMBINED SYSTEM WITH SATELLITE STORAGE

Storm	Storm	Peak Outflow (cfs)				Suspended Solids*			
Frequency	Pattern	0%	9.4%	17%	41.3%**	0%	9.4%	17%	41.3%
5-Year	Intermediate	1407	1201	875	607	1972 (2939)	511 (1935)	511 (1851)	713 (1423)
1-Year	Intermediate	616	358	361	137	2364 (2827)	875 (1789)	825 (1535)	811 (611)
1.3-Month	Intermediate	215	127	128	49	2660 (2215)	1059 (987)	1007 (976)	855 (202)
5-Year	Advanced	766	285	-	-	138 (2993)	92 (1342)	- -	- -
1-Year	Advanced	385	233	232	87	204 (2683)	196 (1559)	178 (1272)	169 (299)
1.3-Month	Advanced	130	79	74	27	448 (2099)	246 (922)	223 (910)	351 (180)
Pounds of Dry Weather Solids Deposit in Sewers						1124	380	380	872

* Numbers not in the parenthesis are peak SS concentration in mg/l discharged. Those in parenthesis are total pounds of SS discharged.

** Storage in percent of 31.5 MG: Storage locations are indicated in Figure 22.

Note: 1 cfs = 0.0283 m^3/sec; 1 lb = 0.453 kg pollution. The cost-effectiveness of providing satellite storage also requires consideration of downstream interceptors, storage and treatment facilities as discussed hereinafter.

POLLUTION CONTROL POTENTIAL OF CONVENTIONAL COMBINED SYSTEMS

Pollution from combined sewage overflows has been abated by installing flow control systems in large trunks or interceptors of existing, well designed, conventional systems to contain peak flows for later discharge to treatment facilities. The Cities of Seattle, Washington, Minneapolis-St. Paul, Minnesota, and Cleveland, Ohio have installed dynamic flow control systems and demonstrated the feasibility of remote, computer directed control. In Seattle, the cost of each flow control device ranged from about $60,000 to $450,000 with $150,000 being a representative value (18).

The above approach originated the concept of the "advanced system" which provides either in-pipe or satellite storage in the design process. TABLE 10 summarizes the total volume in sewer pipes and storage basins for both conventional and advanced sewer systems. That part of the total pipe volume available for storage would be dependent upon the number and/or location of flow control devices, the pipe slopes and sewer network configuration. The trunk sewer of the conventional combined system designed for District A has a total storage capacity of 9.4 mm (0.37 inches). This storage, if fully effective, would contain the total runoff of 6.6 mm (0.26 inches) which is equivalent to a 1.3-month synthetic storm runoff volume (See TABLE 5). The 1.3-month synthetic storm represents a high return frequency storm. By capturing and treating its runoff significant pollution control can be achieved. The conventional combined system designed using SWMM for a 5-year or more return frequency contains as much storage (14.2 mm or 0.56 inches for District A) as the minimum size advanced combined system investigated and could be equally effective for pollution control.

COST COMPARISON OF ALTERNATIVE COLLECTION SYSTEMS

The collection system includes the lateral and trunk sewers and storage facilities within each drainage district. House connections are practically the same for all alternatives and were therefore not considered in making comparison.

The number of manholes was estimated assuming a uniform spacing interval of 91 m (300 feet). For excavation depths of less than 8.5 m (28 feet), the estimated unit cost was $2,500 and of more than 8.5 m (28 feet), $5,000. The excavation depth of a manhole is measured as the average excavation depth of the sewer it serves.

The cost of sewers is computed in accordance with the unit prices of Figure D-15 or TABLE D-17. The excavation depth was determined as the average of the depths of the upstream and downstream ends of the sewer at the junction manholes. The sewer depth is determined by its invert elevation. At a junction manhole, where more than one upstream sewer connects, the lowest crown elevation of the upstream connecting sewers was taken as the

TABLE 10. VOLUME* IN SEWER PIPES AND STORAGE BASINS, DISTRICT A

Description	Lateral Sewer (inches)	Trunk Sewer (inches)	Total (inches)
Separate Storm	-	-	0.39
Conventional Combined	0.19	0.37	0.56
8.3% In-Pipe	0.28	0.28	0.56
16.5% In-pipe	0.41	0.23	0.64
25.0% In-pipe	0.55	0.18	0.73
41.3% In-pipe	0.83	0.15	0.98
9.4% Satellite	0.19	0.43**	0.62**
17.0% Satellite	0.19	0.61**	0.80**
41.3% Satellite	0.19	0.94**	1.13**

* Total Volume expressed as inches of runoff over the entire watershed of District A.

** Includes satellite storage volume. Note: 1 inch = 25.4 mm crown of the downstream sewer to avoid surcharging upstream sewers above their crown when the downstream sewer flows full or near full. The minimum sewer cover was 1.8 m (6 feet). Reduction of the minimum cover to 1.52 and 1.22 m (5 and 4 feet) would reduce the cost of District A sewer system by about five and eight percent respectively.

A computer program was written to estimate sewer system cost. The same computer program was used for all sewer system alternatives including the separate storm system. Although the assumption of matching the downstream sewer crown with the lowest of the upstream sewer crowns would somewhat increase the cost of the separate storm sewer system, the increase should not alter the conclusion.

Cost of Conventional Sewer Systems

TABLE 11 summarizes the costs of alternative conventional separate and combined sewer systems in District A. Compared are the following alternative systems:

1. A conventional combined system with a minimal amount of overland flow permitted for urban runoff. In this system, all sewers would carry combined sewage except those at the extreme upstream end of a sewer branch. These upstream sewers would convey sanitary flow only as previously described (Case 1).
2. A combined system with the same amount of overland flow as a separate storm sewer system. The total length of sewers carrying combined sewage is the same as the toal length of separate storm sewers (Case 2).
3. Separate sanitary and storm systems with storm sewers surcharged. In this alternative, a significant amount of overland flow is permitted to reduce the length of storm sewers required (Cases 3 plus 4).
4. A separate sanitary and storm system with storm sewers designed without surcharge (Cases 3 plus 5).

The costs summarized in TABLE 11 indicate:

1. A conventional combined system (Case 1) would cost about the same as a separate system with surcharge (Cases 3 plus 4).
2. The combined system permitting significant overland flow (Case 2) is the least-cost system with an estimated cost about five percent lower than the cost of Case 1 and Cases 3 plus 4.
3. The most expensive system is Cases 3 plus 5 with a cost of about 13 percent greater than for Case 1 and Cases 3 plus 4.
4. Surcharging storm drains permits reducing their costs by about 20 percent.

TABLE 11. COST COMPARISON OF CONVENTIONAL COMBINED AND SEPARATE SEWER SYSTEMS ($$10^6$), DISTRICT A

Case No.	System Type	Sewers Carrying Comnined Sewage	Sewers Carrying Sanitary Flow Only	Total Cost**
1.	Combined	$15.91 (63,640)*	$2.49 (42,360)	$18.40 (106,000)
2.	Combined with the same degree of overland flow as separate storm	$12.39 (29,000)	$4.46 (77,000)	$17.05 (106,000)
3.	Sanitary	---	$8.12 (106,000)	$ 8.12 (106,000)
4.	Separate Storm designed with surcharge	---	---	$ 9.88 (29000)
5.	Separate Storm designed without surcharge	---	---	$12.39 (29,000)

* Numbers in parenthesis are length of sewer system in feet.

** Including cost of sewer pipes and manholes

For urban areas requiring storm and sanitary sewers, a combined sewer system (Case 2) would be more economical than separate systems with respect to the cost of collection sewers. For areas where the existing sewers can be used to convey sanitary wastes, as in District A, sewer separation is the least-cost alternative (Case 4 as compared to case 2). However, where an existing system contains substantial capacity to convey part of the storm flows, it may be desirable to intercept in a separate system only sufficient runoff to prevent frequent flooding. As previously discussed, combined systems, designed as described in this report, also offer advantages for pollution abatement because of the storage that can be made available in the pipes.

In the subsequent cost comparisons of conventional systems with advanced combined systems, Case 1 is used since the configuration of the advanced combined system was compatible with this case.

Cost of Advanced Sewer Systems

TABLE 12 presents a cost comparison of combined systems with various amounts of either in-pipe storage provided in upstream lateral sewers to achieve greatest economy or satellite storage along the trunk sewers. The costs of collection sewers for the entire City were projected from District A sewer costs by the ratio of drainage areas. These costs do not include interceptors, downstream storage and treatment facilities. Storing about 8.3 percent of the overland runoff from a 5-year synthetic storm in the lateral sewers would result in the least cost for trunk and lateral sewers in District A. The increased cost of lateral sewers is more than offset by the reduced cost of trunk sewers. However, while there is a small increase in cost (about $1.1 and $1.9 million, respectively) as the storage volume is doubled and tripled, there is a substantial reduction of peak outflow which the downstream interceptors and subsequently downstream storage and treatment facilities may be required to accept. Depending on specific circumstances, this increase on collection system costs may be offset by capital investment required for downstream sewer facilities. For a combined system with satellite storage, the potential cost savings of providing storage may be greater than for a system with in-pipe storage depending on the unit cost and location of the storage. In TABLE 12, sewer system costs are presented with unit costs equal to $0.08 to $0.26 per liter ($0.30 and $1.00 per gallon) of storage. This cost could vary depending upon the type of storage basin required and the environmental restraints. With respect to District A, a satellite storage capacity of 20,800 m^3 (5.5 million gallons) or about 17 percent of 5-year synthetic storm runoff should result in the least cost in the three cases compared if the unit storage cost is $0.08 per liter ($0.03 per gallon) of storage. At a unit cost of $0.26 per liter ($1.00 per gallon) of storage, the cost of collection sewer system increases with an increase in storage capacity.

TABLE 12. COST SUMMARY OF COMBINED COLLECTION SYSTEMS WITH IN-SYSTEM STORAGE

Case No.	System Description	Total Volume Stored In Million Gallons	Cost ($$10^6$)					Peak Outflow from Dist. A (cfs)
			Lateral Sewer	Trunk Sewer	Storage Basin	Dist. A* Total	City** Total	
1.	Conventional Combined	0	8.30	9.20	0	18.40	122.29	1407
2.	8.3% In-pipe	2.67	9.55	7.29	0	17.74	117.90	1096
3.	16.5% In-pipe	5.33	11.51	6.38	0	18.79	124.88	788
4.	25.0% In-pipe	8.0	13.68	5.07	0	19.65	130.60	607
5.	41.3% In-pipe	13.34	18.72	4.43	0	24.05	159.84	448
6.	9.4% Satellite	3.01	8.30	8.43	(0.90) (3.01)	(18.53) (20.64)	(123.16)*** (137.11)****	1208
7.	17.0% Satellite	5.50	8.30	6.77	(1.66) (5.50)	(17.63) (21.47)	(117.17) (142.70)	879
8.	41.3% Satellite	13.37	8.30	5.87	(4.01) (13.37)	(19.08) (28.44)	(126.81) (189.02)	607

* Includes manhole cost of $0.90 million

** Obtained from District A total by area ratio (4354/655).

( ) *** Cost based on $0.08 per liter ($0.30 per gallon) of storage

( ) **** Cost based on $0.26 per liter ($1.00 per gallon) of storage

Note: 1 million gallons = 3785 m^3
1 gallon = 3.785 liters

SECTION IX
DESIGN AND COST COMPARISON OF ALTERNATIVE CITY-WIDE SEWER SYSTEMS

An objective of this study was the evaluation of the cost-effectiveness of alternative sewer system designs for a given level of pollution control. For the various alternative collection systems developed (a total of 9), the level of pollution control would determine the requirements for (a) interception and treatment of runoff and combined sewage before discharge and (b) capital investment required for interceptors, downstream storage and treatment facilities. Since the hydrographs and pollutographs generated for the alternative collection sewers are different, the downstream system facilities to achieve the same level of pollution control can also be different. That alternative which achieves the desired level of pollution control with the least total cost for collector, interceptor, storage and treatment systems, is the cost-effective system.

Examination of the outflow characteristics of the various synthetic storms from the alternative collection systems provided insight to the interception and treatment requirements. The effectiveness in pollution abatement of varying amounts of interceptor capacity and upstream storage was evaluated using real rainfall events over a period long enough to provide statistically significant information.

Near total elimination of runoff pollutants from District A associated with runoff from the 5-year synthetic storm with intermediate pattern would require the first 90 minutes of runoff from a conventional combined system to be captured and conveyed by interceptors to storage and treatment facilities (Figures 3 and 4). The costs of systems meeting this severe criterion were estimated.

SYNTHETIC STORM RUNOFF

City-wide planning requires development of runoff hydrographs and pollutographs for each of the City's drainage districts for the storms of interest to develop the required interceptor and treatment storage facilities. The necessary hydrographs and pollutographs for each drainage district were derived from information developed in the detailed study of District A.

The two primary factors governing the shape and peak flow rate of the hydrograph for a given drainage basin are area and percent of imperviousness. The area affects the peak flow rate, the flow duration and the peaking time of the hydrograph. The percent of imperviousness, which is dependent on land use, affects the runoff volume and the peak flow rate. Study of 5-year synthetic storm hydrographs computed with SWMM for various areas within District A indicated that hydrographs are relatively similar in character within a reasonable range of variation in drainage area and percent of imperviousness, and that dividing the volume by the peak runoff rate yields a relatively constant value in time units. Such hydrographs may be "normalized" by equating the defining ordinates to values based on Q/Q_{peak} where Q is the flow rate at given time and Q_{peak}, the peak flow rate. The runoff volume for the synthetic storm was estimated as in STORM. With the hydrograph constant determined from the data developed for District A, the peak flow was obtained by dividing the volume by the hydrograph constant and the values of the 5-year synthetic storm hydrograph obtained by multiplying the peak flow by the ordinates of the appropriate "normalized" hydrograph. The maximum ordinate is equal to 1 (Q_{peak}/Q_{peak}) and the hydrograph constant equals the area of the normalized hydrograph. Five normalized hydrographs were developed with data as summarized in TABLE 13. The normalized hydrograph No. 1 is based on the hydrographs for sewer elements numbers 205, 209, 219 and 223 with drainage areas of 13.2, 21.8, 58.3 and 21.0 hectares (32.5, 53.9, 144.0 and 52.0 acres) respectively. The corresponding percents of imperviousness of the areas drained to these elements are respectively 60.0, 55.3, 53.5, and 43.0. The estimated peak flow of 3.14 m^3/ sec (111.0 cfs) for sewer element number 205 was obtained by dividing the runoff volume of 634 m^3 (22,400 cubic feet) by the hydrograph constant of the normalized hydrograph No. 1 in seconds.

The estimated peak flow for element numbers 209, 219 and 223 were computed in the same manner. The agreement between the peak flow computed with SWMM and that computed using the normalized hydrograph, indicates the percent of imperviousness affects the runoff volume more than the hydrograph shape within the range of variation considered. For other normalized hydrographs, the agreement between the estimated peak flow and that obtained from SWMM is also good. The adjustment of the time of peak flow occurrence with respect to District A is listed in TABLE 14 for four ranges of drainage areas. The peak outflow from District A occurs 74 minutes after the start of the 5-year synthetic storm. Figure 27 presents two of the normalized hydrographs, one for a drainage area greater than 215 hectares (530 acres) and one for area less than 60 hectares (150 acres). The hydrograph for the larger drainage area has a greater spread and later peaking time. The peaking time of the other normalized hydrographs fall between the two peaks shown in Figure 27.

The 5-year synthetic storm runoff volume in drainage districts other than District A were computed using the calibrated runoff coefficients of 0.55 for the pervious area and 1.0 for the impervious area. TABLE 15 summarizes the relevant data involved in the computation of outflow hydrographs for 24 drainage districts. Runoff from District Y drains toward the City of Newark, and is not served by the Easterly or Westerly Interceptor.

Figure 28 shows the outflow hydrographs for Districts A, E and H respectively. To develop outflow hydrographs for drainage districts other

TABLE 13. SUMMARY OF NORMALIZED HYDROGRAPH DEVELOPMENT FROM DISTRICT A DATA

Normalized Hydrograph No.	Hydrograph** Constant (seconds)	Applicable Range of Area (Acres)	Sewer Element Number	Tributary Area (Acres)	% Imp.	Runoff* Volume (10^6ft.)	Peak SWMM	Flow (cfs) Estimated
1.	2021.2	150 or less	205	32.5	60.0	.224	118.4	111.0
			209	53.9	55.3	.362	178.2	178.0
			219	144.0	53.5	.976	476.8	482.9
			223	52.0	43.0	.308	151.8	152.2
2.	2340.8	150–250	324	215.0	43.0	1.441	615.0	615.0
3.	2440.1	250–350	334	290.0	43.0	1.891	778.2	775.3
			338	340.0	43.0	2.204	898.3	903.4
4.	2779.0	350–530	350	380.0	44.0	2.450	895.4	880.0
			352	530.0	50.0	3.440	1220.9	1238.0
5.	2978.6	530 or more	364	655.1	47.0	4.160	1406.9	1406.9

* Runoff volume obtained from SWMM output for District A

** Runoff volume/Q_{peak} = Hydrograph Constant
Runoff volume/Hydrograph Constant = Q_{peak}

TABLE 14. ADJUSTMENT OF PEAKING TIME WITH RESPECT TO THAT OF DISTRICT A

Range of Drainage Area A (Acres)	Peaking Time Ahead of District A (minutes)
A <70	8
70 ≤A <250	6
250 ≤A <500	4
500 ≤A	0

Note: 1 acre = 0.405 hectares

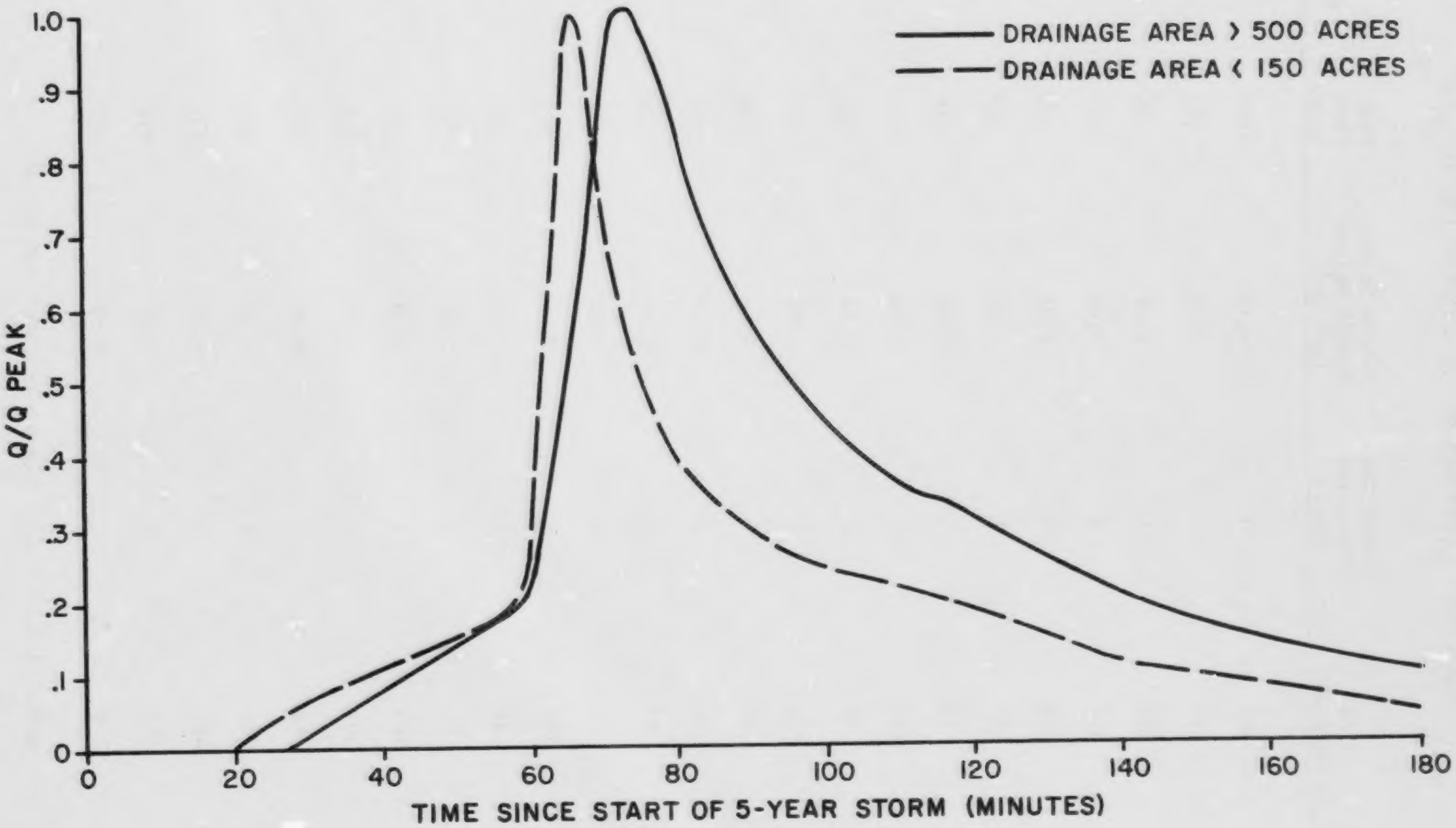


Figure 27. Normalized hydrographs

TABLE 15. DEVELOPMENT OF OUTFLOW HYDROGRAPHS

Drainage District	Area (acres)	Normalized Hydrograph Used	5-Year Storm Runoff Volume (10^6 ft^3)	Peak Flow (cfs)	Downstream Inlet
A	655	5	4.18	1407	204
B	112	1	0.71	351	200
C	111	1	0.76	376	202
D	117	1	0.81	401	202
E	229	2	1.57	671	202
F	88	1	0.61	302	206
G	34	1	0.27	134	206
H	122	1	0.92	455	208
I	70	1	0.48	237	210
J	420	4	2.82	1015	212
K	65	1	0.46	228	210
L	66	1	0.48	237	110
M	67	1	0.44	218	212
N	79	1	0.52	257	212
O	103	1	0.67	331	300
P	37	1	0.28	139	300
Q	83	1	0.60	297	300
R	303	3	2.36	967	300
S	207	2	1.37	555	108
T	62	1	0.44	218	106
U	331	3	2.37	971	104
V	440	4	3.16	1137	102
W	415	4	2.77	997	102
X	138	1	0.85	421	100

Note: 1 acre = .405 hectares; 1 ft^3 =0.283 m^3

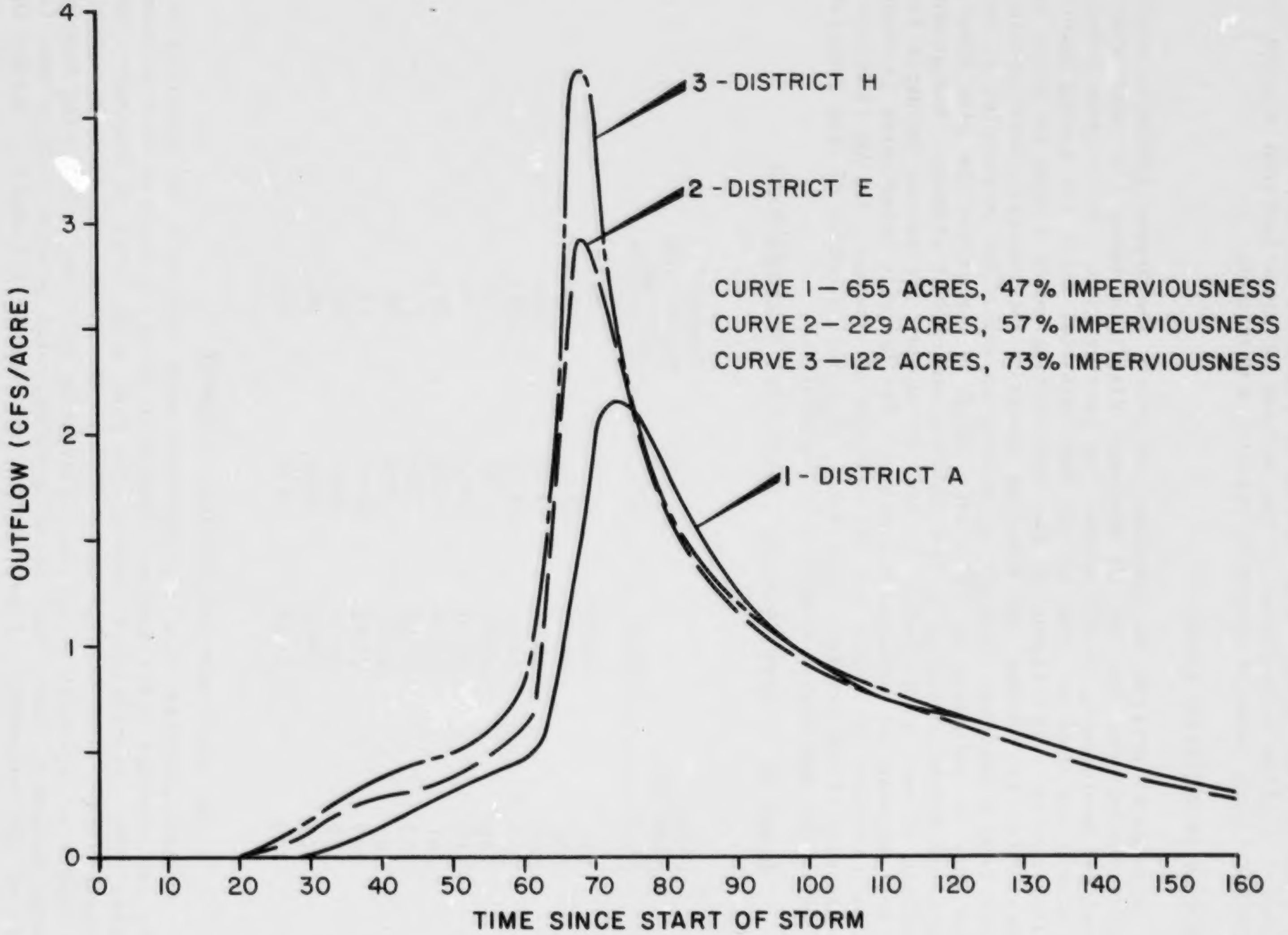


Figure 28. Outflow hydrographs, Districts A, E and H

than District A for systems with in-pipe or satellite storage, the respective ratios of outflow hydrographs of the various advanced combined systems to the conventional combined system in District A were used.

DESIGN OF INTERCEPTING SEWERS

A computer program was developed to obtain the 5-year synthetic storm outflow hydrographs for the 24 drainage districts using the appropriate normalized hydrograph, runoff volume, and peaking time. The program output was stored and used as input to the SWMM TRANSPORT Block for sizing intercepting sewers. The layout of the intercepting sewer system is shown in Figure D-14. It follows the existing Westerly and Easterly Interceptors. Runoff from a drainage district is assumed to enter the interceptor at one inlet only as indicated in TABLE 15. TABLE 16 summarizes the pipe slope, length and ground elevation at the upstream non-conduit element. The ground elevation of each pipe element is used to compute the excavation depth for the cost estimate of intercepting sewers. The Easterly Interceptor is about 6.4 km (4 miles) in length and the Westerly Interceptor, 4.8 km (3 miles). The capacity of the existing interceptor is a small fraction of that required to convey the peak 5-year synthetic storm runoff.

TABLE 16. INTERCEPTING SEWER PIPE ELEMENT DATA

Pipe Element No.	Slope	Length (ft)	Ground* Elevation (ft. MSL)
101	.0005	2517	5.
103	.0005	5769	10.
105	.0007	3982	15.
107	.0005	3125	17.
109	.0005	3470	13.
111	.0010	2950	8.
201	.0010	2500	30.
203	.0015	2500	27.
205	.0007	1000	24.
207	.0002	2000	16.
209	.0050	2000	12.
211	.0010	3000	10.
213	.0018	2970	11.

* at upstream non-conduit element

In the analysis, new interceptors were assumed to be required and only one storage basin and one treatment facility located near element number 300. As mentioned before, the U.S. Army Corps of Engineers has planned several diked storage basins with an estimated total storage capacity of 80,000 m^3 (21 million gallons) near this location. The Joint Meeting Sewage Treatment Plant, now being upgraded, will provide 1.76 m^3/sec (40 mgd) of its wet-weather flow capacity for use by Elizabeth. Hence, the assumption of centralizing storage and treatment facilities is appropriate to the actual condition.

TABLE 17 presents a summary of intercepting sewer sizes for the alternatives investigated defined by the type of collection system and the amount and types of in-system storage. Except for sewers carrying sanitary wastes, all other sewers were sized with the SWMM TRANSPORT Block. Intercepting sewers for the conventional separate system were assumed to have dual pipes in each pipe element, one carrying sanitary waste and the other storm flow. This dual pipe sewer system would be more economical than one pipe carrying both sanitary waste and storm flow because the off-line storage required would be substantially less (TABLE 18). The sewers intercepting sanitary wastes were designed using Equation 1 to determine peak domestic flow plus peak commercial and industrial flows. The peak domestic flow of a drainage district was computed using the district population shown in TABLE D-13. The peak commercial and industrial flows were calculated from land use distribution shown in TABLE D-12 and the assumed rate of 187,000 liters per hectare (20,000 gallons per acre). The intercepting sewer sizes for carrying storm flow only and for combined sewage are the same since the difference of flow rate between the two systems is the sanitary wastewater, which is small in comparison with the 5-year synthetic storm runoff rate.

The sewer sizes required for the advanced combined systems decrease as the amounts of in-system storage increase, since the peaks of outflow hydrograph from a drainage district decrease.

AMOUNT OF STORAGE REQUIRED VERSUS TREATMENT RATE

Ninety-two percent of the SS would be treated if the initial 90 minutes of runoff of the 5-year synthetic storm were captured assuming a conventional combined system is provided in District A. The remaining SS would be discharged untreated to the receiving waters. Since in-system storage modifies the outflow pollutograph characteristics, the duration of runoff to be captured for equal amounts of pollutant overflow varies with the amount of storage provided. In District A, satellite storage capacities equal to 9.4, 17.0 and 41.3 percent of the 5-year synthetic storm runoff would require the capture of the initial 92, 100 and 124 minutes of runoff respectively to capture for the treatment the same amount (92 percent of the total in the 5-year synthetic storm) of SS. The greater duration of runoff to be captured results since the period of significant pollutant spillage from storage basins is extended as the storage volume increases.

For advanced combined systems with various amounts of in-pipe storage, the duration of runoff to be captured was assumed to be the initial 90 minutes for the 5-year synthetic storm. This is the same as for a conventional combined system. In-pipe storage, using a fixed orifice and overflow weir (Figure 16) for control, can be effective during the peak portion of runoff hydrograph starting about 60 minutes after the beginning of the storm. During this period, dry-weather sewer deposits should be fully cleansed and most of the pollutants washed off the streets would be contained in the next 30 minutes of runoff. In-pipe storage would delay somewhat the

TABLE 17. SUMMARY OF INTERCEPTING SEWER DIAMETERS (FEET AND COST ($$10^6$)

Pipe Element No.	Conventional Separate System		Conventional Combined System	In-pipe Storage*				Satellite Storage*		
	Sanitary	Storm		8.3%	16.5%	25.0%	41.3%	9.4%	17.0%	41.3%
101	1.5	10.0	10.0	9.0	7.5	6.5	6.0	8.5	7.0	6.0
103	2.25	17.0	17.0	16.0	15.0	13.0	12.5	16.5	15.0	13.0
105	3.5	17.5	17.5	16.5	14.5	13.0	12.0	17.0	14.5	13.0
107	4.0	18.5	18.5	17.5	15.5	14.0	12.5	17.0	16.0	13.5
109	4.0	18.5	18.5	18.5	16.5	15.0	13.0	18.5	16.5	14.0
111	4.0	17.0	17.0	16.5	14.5	13.0	11.5	16.5	14.5	12.0
201	1.25	7.5	7.5	7.0	6.0	5.5	5.0	7.0	6.0	5.0
203	2.25	12.0	12.0	11.5	10.0	9.0	9.0	11.5	10.5	8.5
205	3.0	19.0	19.0	17.0	15.0	13.5	12.0	17.5	15.0	13.0
207	4.0	23.5	23.5	23.0	17.5	16.0	14.5	22.5	19.0	17.5
209	2.5	13.5	13.5	13.5	12.0	10.5	9.5	13.0	11.0	10.0
211	3.5	19.0	19.0	19.0	16.5	14.5	12.5	18.0	15.5	13.5
213	3.5	18.0	18.0	17.5	15.5	13.5	12.0	16.5	15.0	13.0
Cost ($$10^6$)	10.73	97.57	97.57	87.03	77.86	56.17	46.80	82.01	75.32	52.47

* In percent of peak 5-year synthetic storm runoff volume for District A (119,000 m^3 or 31.5 MG)

Note: 1 foot = 0.3048 meters

TABLE 18. OFFLINE STORAGE REQUIRED (10^6 GALLONS) AND COST FOR EQUAL WEIGHT OF SS DISCHARGED TO RECEIVING WATERS

Case No.	Sewer System	Treatment Rate 40 mgd	 200 mgd	 400 mgd
1	Conventional* Separate	26.67 (7.88)	22.44 (6.73)	18.43 (5.53)
2	Conventional** Combined	84.87 (25.46)	78.32 (23.50)	71.51 (21.45)
3	8.3% In-pipe**	78.80 (23.64)	72.28 (21.68)	65.51 (19.65)
4	16.5% In-pipe**	59.91 (17.97)	53.40 (16.02)	46.63 (13.99)
5	25.0% In-pipe**	49.20 (14.76)	42.63 (12.79)	35.85 (10.76)
6	41.3% In-pipe**	37.66 (11.30)	31.14 (9.34)	24.37 (7.31)
7	9.4% Satellite ***	74.21 (22.26)	67.68 (20.30)	61.70 (18.51)
8	17.0% Satellite +	74.81 (22.44)	67.44 (20.33)	60.33 (18.10)
9	41.3% Satellite++	56.98 (17.09)	48.45 (14.54)	39.62 (11.89)

Number in the parenthesis are storage costs ($\$10^6$) for a unit cost of $0.30 per gallon of storage.

*Capture of runoff starts 30 minutes and ends 70 minutes after the start of storm.

**Capture the initial 90 minutes of runoff.

***Capture the initial 92 minutes of runoff.

+Capture the initial 100 minutes of runoff.

++Capture the initial 124 minutes of runoff.

Note: 1 million gallons = 3785 m^3
1 mgd = 0.0438 m^3/sec

discharge of pollutant mass, but this delay would not be as long as with satellite storage since, under the assumed operating mode, storage would not be utilized until flows in excess of interceptor pipe capacity are experienced.

Runoff from the separate storm system for the 5-year intermediate pattern synthetic storm, as computed by SWMM, for the first 30 minutes is practically pollution-free (Figure 3), and capture of this small portion of runoff (Figure 2), should not be required. The dry-weather flow would be fully captured in the separate system and only stormwater would overflow. Since stormwater is less polluted than the combined sewage, less storage should be required or a greater amount of overflow may be allowed. The treatment of 92 percent of the SS would require storing runoff for a duration of 40 minutes, from 30 to 70 minutes after the start of the synthetic storm.

The amount of storage required would also depend upon the rates of stormwater runoff and combined sewage treatment. Runoff rate, up to the treatment capacity, would bypass the storage to treatment. TABLE 18 presents the appropriate off-line storage capacity for treatment rates of 1.75,8.77 and 17.54 m^3/sec (40,200 and 400 mgd). This range of treatment rates indicates the effect of treatment capacity on downstream storage costs of a city-wide sewer system.

It was assumed that the required off-line storage volume would be provided near element number 300.

The off-line storage volume required generally decreases with the amount of in-system storage provided. The off-line storage volumes required for 9.4 percent and 17.0 percent of satellite storage are about the same since the outflow hydrographs and pollutographs from these two systems are similar. (Figures 22 and 23)

The preceding analysis was calculated using SWMM with STORM default values. The values have not been verified by testing of field samples. However, the relative relationships developed should be adequate for this stage of planning.

COST COMPARISON OF ALTERNATIVE SEWER SYSTEMS

The cost of intercepting sewers was estimated as described for the collection sewers. Sewer sizes greater than 4.3 m (14 feet) in diameter were replaced by multiple parallel sewer pipes with sizes less than 4.3 m (14 feet) while maintaining the same total cross-sectional area. For example, a 5.8 m (19 feet) diameter pipe as required for the pipe element number 205 in a conventional combined system was replaced by two 4.1 m (13.5 feet) diameter pipes; and one 7.2 m (23.5 feet) diameter pipe for element number 207 was replaced by three 4.1 m (13.5 feet) diameter pipes. This enabled the use of the unit costs shown in Figure D-15 and TABLE D-17.

The cost of the intercepting sewers for the alternative systems are shown in TABLE 19. The cost of intercepting sewers for the advanced combined

TABLE 19. TOTAL COST OF CITYWIDE SEWER SYSTEMS

Case No.	Sewer System	Collection System Cost (\$$10^6$)	Interceptor Sewer Cost (\$$10^6$)	Total Sewer System Cost (\$$10^6$)			
				40 mgd*	200 mgd*	400 mgd*	Average
1	Conventional Separate	138.31	108.30	(255.56) (274.35)	(256.81) (272.52)	(258.85) (271.75)	(257.07)** (272.87)***
2	Conventional Combined	122.29	97.57	(246.39) (305.80)	(246.83) (301.65)	(248.02) (298.08)	(247.08) (301.84)
3	8.3% In-pipe	117.90	87.03	(229.64) (284.80)	(230.08) (280.68)	(231.29) (277.15)	(230.34) (280.88)
4	16.5% In-pipe	124.88	77.86	(221.78) (263.72)	(222.23) (259.61)	(223.44) (256.08)	(222.48) (259.80)
5	25.0% In-pipe	130.60	56.17	(202.60) (237.04)	(203.03) (232.87)	(204.24) (229.33)	(203.29) (233.08)
6	41.3% In-pipe	159.84	46.80	(219.01) (245.37)	(219.45) (241.25)	(220.66) (237.72)	(219.71) (241.45)
7	9.4% Satellite	(123.16)** (137.11)***	82.01	(228.50) (294.40)	(228.94) (290.27)	(230.39) (287.53)	(229.28) (290.73)
8	17.0% Satellite	(117.17) (142.70)	75.32	(216.00) (293.90)	(216.19) (288.93)	(217.30) (285.06)	(216.50) (289.30)
9	41.3% Satellite	(126.81) (189.02)	52.47	(197.44) (299.54)	(197.29) (293.41)	(197.88) (287.82)	(197.54) (293.59)

*Costs of primary settling are $1.07, 3.47 and 6.71 million dollars respectively.(6)
**Based on unit storage cost of $0.08 per liter ($0.30 per gallon)
***Based on unit storage cost of $0.26 per liter ($1.00 per gallon)

Note: 1 mgd = 0.0438 m^3/sec

systems is less than for the conventional separate or combined systems. The cost of off-line storage was estimated using unit costs of \$0.08 and \$0.26 per liter (\$0.30 and \$1,00 per gallon) of storage provided. TABLE 18 uses only a storage cost for \$0.08 per liter (\$0.30 per gallon) of storage. Capital cost of treatment for facilities was estimated using the SWMM STORAGE /TREATMENT Block. The treatment processes were assumed to include bar racks and microstrainers. Sedimentation is assumed to occur in the storage basins. This essentially conforms to the contemplated operation of the Joint Meeting Plant since peak wet-weather flows are proposed to bypass the biological treatment units. The unit cost of the treatment processes and land, and the amortization period and interest rate used, were the default values provided in SWMM. The computed costs were converted to the ENR construction cost index of 1800. The computed capital costs of treatment facilities were \$1.07, \$3.47 and \$6.71 million respectively for treatment capacities of 1.75, 8.77 and 17.54 m^3/sec (40, 200 and 400 mgd). If more sophisticated treatment is required, these costs could increase by about 40 times and would favor larger basins and lower treatment capacities.

TABLE 19 presents the total cost of the nine alternative sewer systems including collection sewers, in-system storage, intercepting sewers, off-line storage and treatment facilities. Two cost totals are given, one assumes a storage cost of \$0.08 per liter (\$0.30 per gallon) and the other \$0.26 per liter (\$1.00 per gallon). For each alternative, the total cost of the sewerage system is about the same for the minimum treatment assumed regardless of treatment rate. However, if more sophisticated treatment is required (such as activated sludge at the Joint Meeting Plant), the lower treatment rate would provide substantial cost advantage. The total costs are presented in Figure 29 for the various amounts of in-pipe and satellite storage. Points A and B are a conventional separate system computed with a unit cost of off-line storage equal to \$0.08 and \$0.26 per liter (\$0.30 and \$1.00 per gallon), respectively. The following conclusions can be drawn:

1. In areas where the cost of off-line storage is relatively high, sewer separation would be more economical than a conventional combined system. On the other hand, when storage can be economically provided, a conventional combined system would be more economical than a conventional separate system.

2. The maximum amount of in-pipe storage that may be optimally provided is about 30 percent of the 5-year synthetic storm runoff, within the range of costs assumed for off-line storage.

3. The conventional separate system would be the most expensive system if storage (off-line and satellite) can be provided at a cost of \$0.08 or less per liter (\$0.30 or less per gallon).

4. At a storage cost of \$0.26 per liter (\$1.00 per gallon), the conventional separate system would be more economical than an advanced combined system with any amount of satellite storage provided.

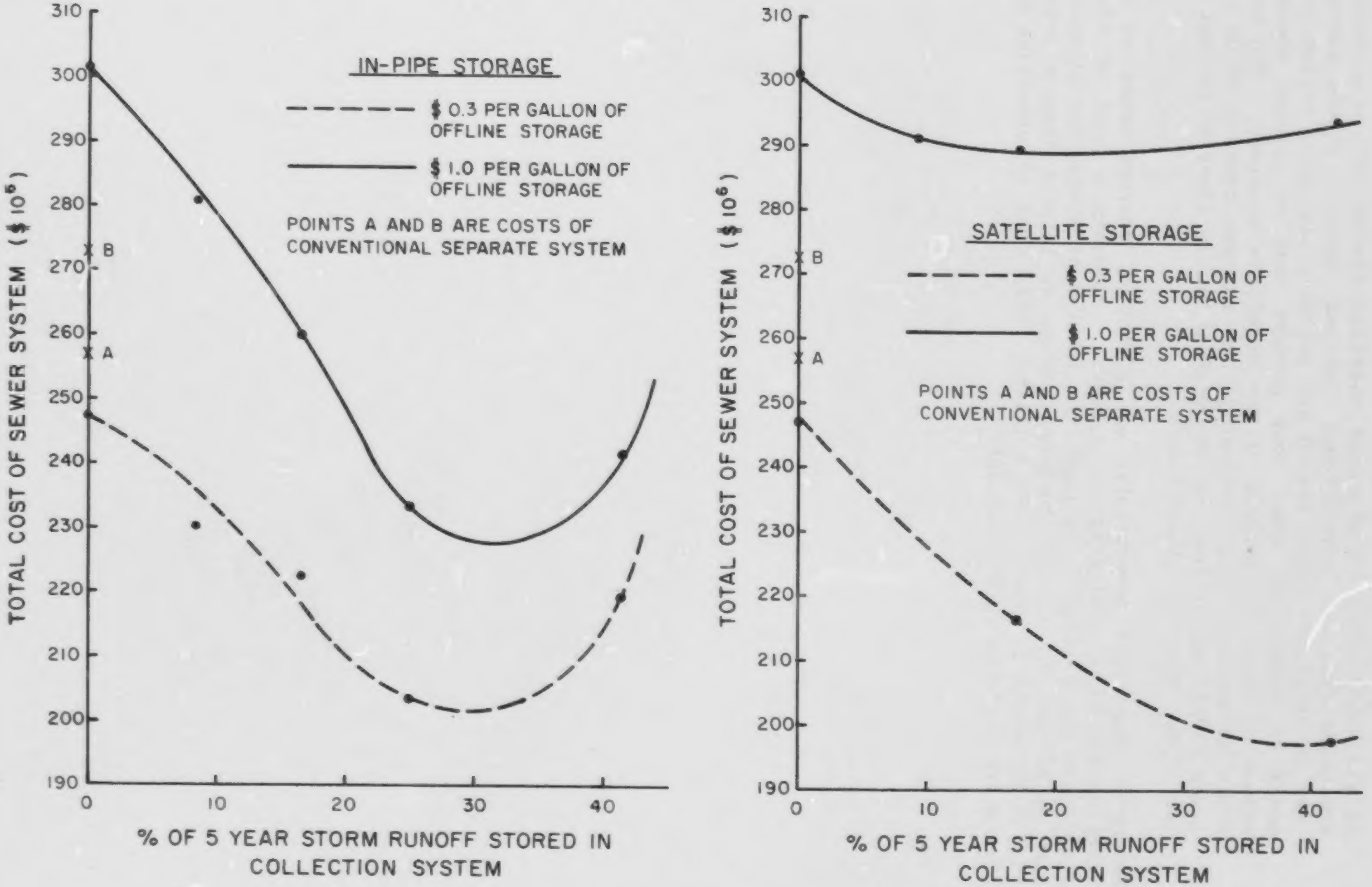


Figure 29. Total cost comparison-city sewer system, in-pipe vs. satellite storage

5. In all instances, an advanced combined system offers economic advantage over the conventional combined system. If the storage cost is $0.26 per liter ($1.00 per gallon), the total system cost would be reduced by about four percent when an optimal storage volume of about 20 percent of the runoff is provided. The cost would be reduced by 20 percent if the storage cost is $0.08 per liter ($0.30 per gallon) and the storage volume about 40 percent of the 5-year synthetic storm runoff.

The above comparison demonstrates the effect of storage costs on the cost-effectiveness of the alternative systems. Assuming a cost of $0.08 per liter ($0.30 per gallon) of storage, the least-cost system for Elizabeth for treating 92 percent of the combined sewage pollutants from a 5-year synthetic storm would be about $200 million. This cost is excessive and alternatives are explored in Section XII.

SECTION X

ANALYSIS OF OVERFLOW QUANTITY AND QUALITY

In previous analyses, collection sewers, storage basins and interceptors have been developed to essentially provide treatment for about 92 percent of the computed pollutants in runoff and municipal wastes resulting from a long return interval, synthetic, intermediate pattern storm hyetograph. Pollutants in combined sewage are subject to random hydrometeorological fluctuations. Use of the same "design storm" for pollution and flood control implies the desired degree of water quality protection can be equated to a level of flood protection in an urban area. This is not valid. The severity and frequency of critical storms for the planning of facilities for control of urban flooding and of pollution is different. While the total pounds of pollutants contained in the runoff from a 5-year synthetic storm is greater than in the runoff from individual frequent storms, the total pollution generated by the frequent storms is greater when weighted by the number of expected occurrences per year. The concentration of pollutants in a frequent storm may also be greater. In the Elizabeth area, about 93 percent of the precipitation events have a gross rainfall amount of less than 25 mm (1.0 inch) with the accompanying street pollutant washout aggregating more than 73 percent of the total.

To develop reasonable criteria for pollution control, the characteristics of runoff quantity and quality from frequent storms (1.3 and 0.35 month return frequencies) were evaluated quantitatively using SWMM. Overflow control effectiveness was also studied, using STORM, to evaluate (a) the dynamic response of the catchment to a series of historical rainfall events, (b) the consequences (overflow quantity and quality) of such events, and (c) the results in terms of an average performance characteristic. The benefits of a combination of storage and treatment (or interceptor) capacity were also comparatively quantified in terms of overall performance under the variety of hydrometeorological conditions experienced in nature. The benefits were based on the characteristics of overflow quantity and quality, using long-term precipitation records. Statistical analysis developed the probable number of annual overflow events, their total volume and their discharge quality (total pounds of pollutants) for 32 alternatives.

Treatment rate, as normally referred to in STORM, was used as interceptor capacity. Overflow from each drainage district was controlled by the interceptor capacity selected. Because storage basins are possible near the treatment plant for flow equalization, treatment rate and interceptor capacity could be different.

Since STORM considers pollutant accumulation on streets, it was used to evaluate the effectiveness of street sweeping practices for source control. Data, developed for District A, was used to determine the effectiveness of overflow control measures by extension to other drainage districts in the City. Simulation of urban runoff from a separate system was made by specifying zero storage and bypassing the dry-weather flow (which would be entirely diverted to treatment) routines. Figure 30 presents the annual amount of SS in street washout and combined sewage overflows versus the amount of annual rainfall. More pollutants should be removed from streets and greater pollutant loads discharged in combined sewage overflows during a wet year than a dry year.

EFFECT OF STORAGE AND INTERCEPTOR CAPACITY

On the basis of a continuous 12-year precipitation record at the Newark International Airport and an assumed capture rate for treatment equal to the peak dry-weather flow, 65.8 combined sewage overflow events should be expected annually from a conventional combined sewer system. Yearly overflows should have lasted a total of more than 3.2 percent of the time. The yearly total overflow volume should have equaled 343 mm (13.5 inches) over the entire watershed of District A or 908,000 m^3 (240.0 million gallons). Based on STORM default values, the pollutant overflow yearly should equal 23,300 kg (51,300 pounds) of SS and 11,000 kg (24,300 pounds) of BOD. All sanitary wastes were assumed to be treated during dry periods.

TABLE 20 summarizes the results of computer simulations over a 12-year period for various combinations of storage capacity and rates of interception for treatment. The number of overflow events, their volume and the mass discharges of SS and BOD are listed. Computer runs were made for storage capacities ranging from 0 to 50,300 m^3 (0 to 13.3 million gallons) and interceptor capacities from 0.10 to 40 m^3/sec (2.4 to 910 mgd). Case No.1 in TABLE 20 represents discharge from a conventional separate storm system.

Figures 31 and 32 present the annual average number of overflow events and mass SS discharged for various combinations of storage volume and interceptor capacity. The number of overflow events and the amount of pollutants discharged could be controlled by increasing storage or interception rate or a combination of both. For a given storage volume, an increasingly greater interceptor capacity would be required to attain each successive increment of pollutant reduction. This diminishing return also applies, for a given interceptor capacity, to storage volume.

The amount of SS and BOD contained in street washout in District A is estimated as 15,300 and 2,600 kg (33,800 and 5,800 pounds) respectively (Case No. 1 of TABLE 20) based on the STORM default values for computing pollutants from dust and dirt washed off the street. This much SS and BOD would reach the receiving waters through a separate storm system with no treatment. With an assumed interception rate of 0.26 m^3/sec (6.0 mgd), the peak DWF, a conventional combined system would annually discharge about 23,300 kg (51,300 pounds) of SS and 11,000 kg (24,300 pounds) of BOD (Case No. 3). These amounts are substantially greater than those estimated for

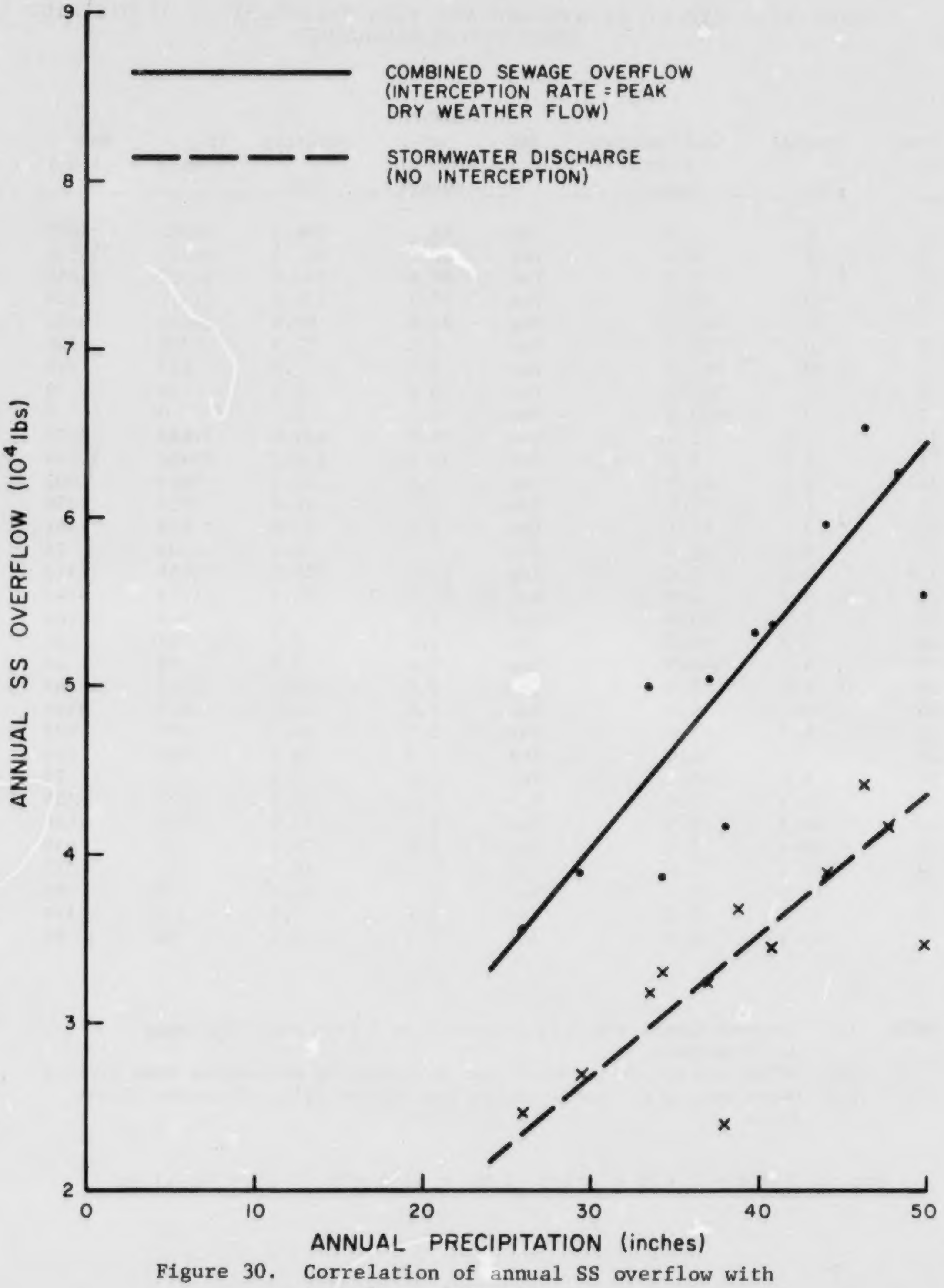


Figure 30. Correlation of annual SS overflow with annual precipitation.

TABLE 20. EFFECT OF STORAGE AND FLOW INTERCEPTION OF DISTRICT A SEWER SYSTEM DISCHARGES

Case No.	Storage (MG)	Intercepting Rate (mgd)	DWF	Number of Overflow Events	Quantity (MG)	SS (lbs.)	BOD (lbs.)
1	0	0	No	69.5	298.7	33822	5803
2	0	2.4	Yes	68.5	285.5	68242	35528
3	0	6.0	Yes	65.8	240.0	51337	24345
4	0	22.6	Yes	40.3	121.0	21387	7138
5	0	41.0	Yes	22.6	69.4	11335	3113
6	0	98.9	Yes	6.6	21.3	2702	620
7	0	161.0	Yes	2.3	7.8	611	149
8	0	355.0	Yes	0.2	0.9	23	9
9	0	910.0	Yes	0.	0.	0	0
10	2.7	2.4	Yes	28.0	185.0	37631	18573
11	2.7	6.0	Yes	24.4	138.7	23466	10199
12	2.7	41.0	Yes	7.8	34.5	3665	1002
13	3.0	69.8	Yes	3.3	16.4	1215	336
14	3.0	82.7	Yes	2.7	13.0	859	242
15	3.0	140.9	Yes	1.1	5.3	203	73
16*	5.3	2.4	Yes	16.0	131.6	21658	11315
17	5.3	6.0	Yes	13.1	90.4	11913	5542
18	5.3	69.8	Yes	1.6	11.0	449	169
19	5.3	82.7	Yes	1.4	8.7	313	126
20	5.3	140.9	Yes	0.6	3.7	92	46
21	8.9	2.4	Yes	9.0	90.2	12962	7588
22	8.0	6.0	Yes	7.8	63.5	6670	3592
23	8.0	22.6	Yes	3.3	26.5	1355	622
24	8.0	41.0	Yes	1.8	14.4	468	234
25	8.0	88.6	Yes	0.8	5.5	122	75
26*	13.3	2.4	Yes	5.0	62.1	6373	4109
27	13.3	6.0	Yes	3.7	35.0	2345	1496
28	13.3	12.9	Yes	1.8	23.7	995	630
29	13.3	22.6	Yes	1.3	16.2	451	310
30	13.3	31.7	Yes	1.1	11.6	268	192
31	13.3	41.0	Yes	0.8	7.8	162	119
32	13.3	69.8	Yes	0.5	4.1	83	61

NOTE:
(a) Assumed Street Sweeping frequency is 7 days and efficiency is 75 percent.
(b) STORM default values were used in computing pollutants from streets
(c) *Runs made with precipitation data (1965-1971), otherwise (1963-1974)

Note: 1 mgd = 2.628 m^3/min; 1 MG = 3785 m^3; 1 lb = 0.453 kg

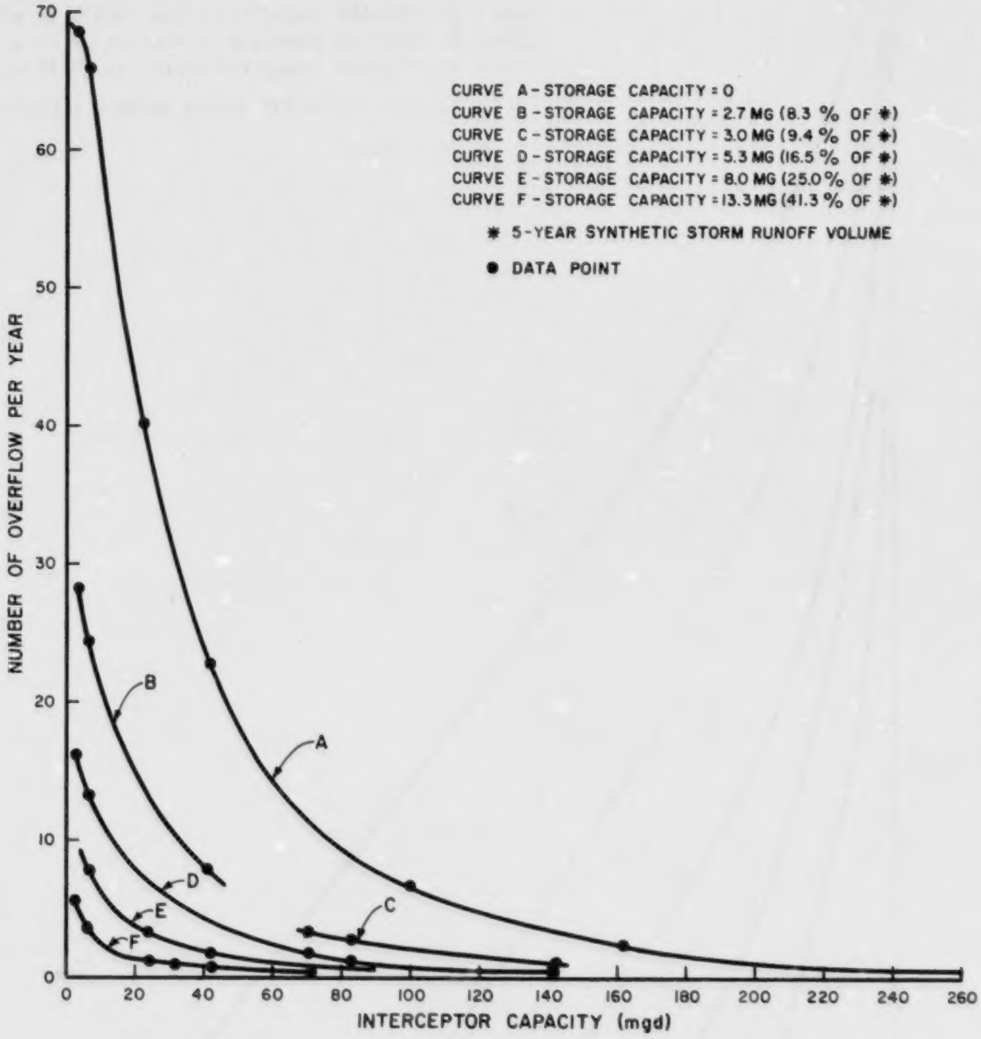


Figure 31. Annual number of overflow events for various storage and interceptor capacities

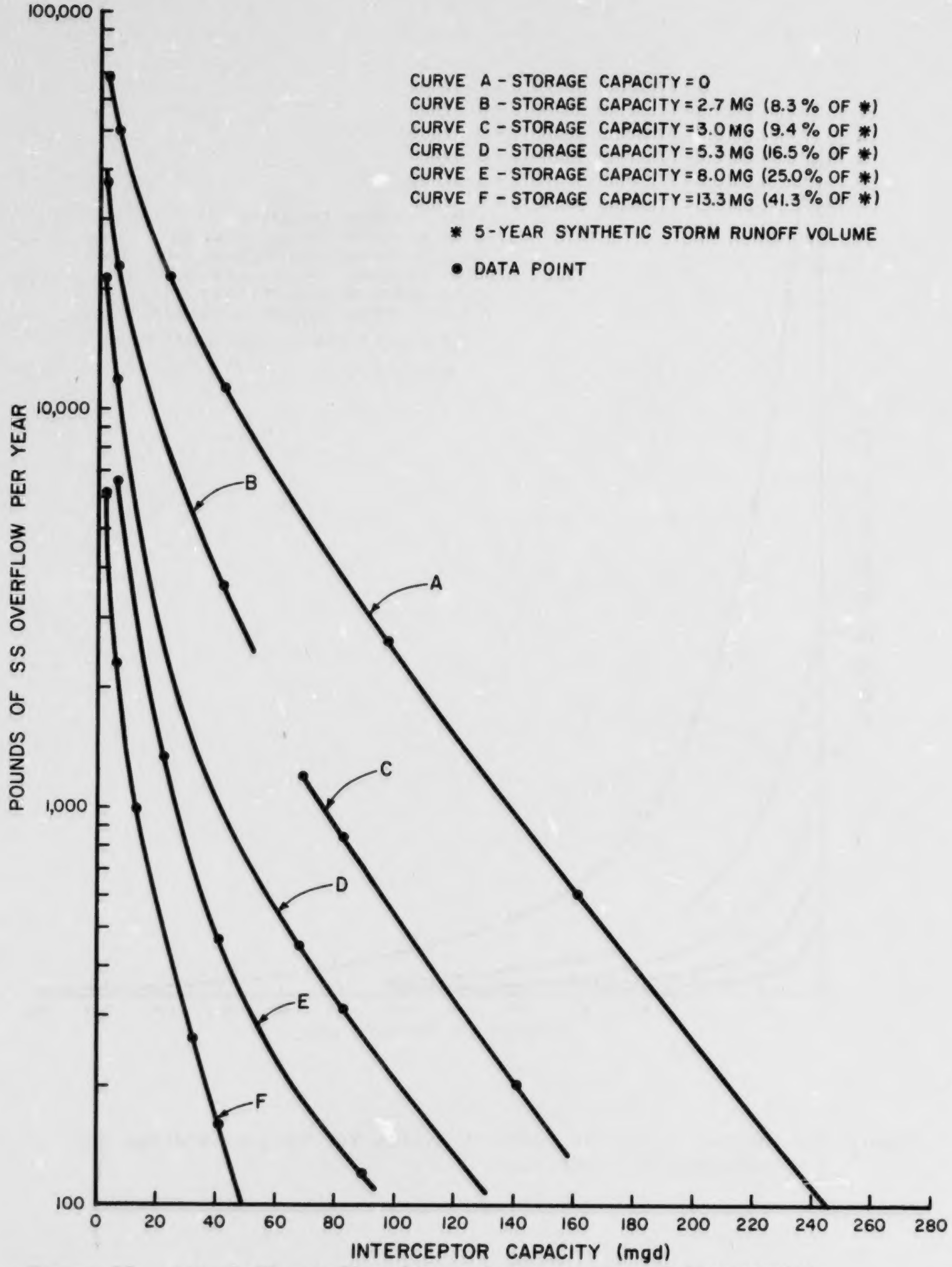


Figure 32. Annual SS overflow for various storage and interceptor capacities

a separate system indicating the pollutant contribution of DWF. The sanitary waste lost through overflows is estimated to be about two percent of the total. Since overflows may be experienced about 3.2 percent or more of the time, it appears that as much as 60 percent of the sanitary wastes might be lost during overflow periods. These percentages were determined from the estimated total weight of annual SS and BOD contained in the sewerage wastes. The high percent of sanitary waste lost in the overflows, coupled with the fact that most of the loss occurs during the frequent rainfalls and is accompanied by high concentrations of pollutants, makes their control significant to upgrading receiving water quality.

To reduce the SS in the overflow from District A by two orders of magnitude, say to 277 kg (611 pounds) per year (Case No. 7 of TABLE 20), it would be necessary to upgrade interceptor capacity to 7 m^3/sec (161 mgd) or to about 18 percent of the peak 5-year synthetic storm runoff estimated for District A. The same amount of SS reduction can be accomplished if 50,300 m^3 (13.3 million gallons) of storage is provided and the interceptor capacity is equal to about 0.9 m^3/sec (20 mgd). To reduce SS in the overflow by one order of magnitude, an interceptor capacity of about 2.7 m^3/sec (62 mgd) would be required. Alternatively, the same reduction should result from upstream storage of 36,300 m^3 (8.0 million gallons) with interceptor capacity (Case No. 22 of TABLE 20) equal to the peak dry-weather flow. Hence, combinations of storage and interceptor capacity could be used to advantage in meeting pollution control objectives.

Data in TABLE 20 permit a quantitative evaluation of the effectiveness of various pollution control measures.

EFFECT OF STREET SWEEPING PRACTICE

Dust and dirt accumulated on streets during dry days contributes to water pollution. A source control alternative is more frequent and effective street sweeping. Computer runs, summarized in TABLE 20, assume a street sweeping interval of seven days and a sweeping efficiency of 75 percent. The reported removal efficiency of the dust and dirt averages about 50 percent and improvement to 70 percent and possibly as high as 90 percent may be feasible under suitable conditions if the parked car problem can be surmounted (19).

TABLE 21 presents the simulation results, using STORM, for various street sweeping efficiencies and intervals. The average quantity of SS in street washout is plotted in Figure 33 for various cleaning intervals and appears almost linearly proportional to the street sweeping interval. If the sweeping efficiency is reduced to 0.5, the amount of pollutants expected would increase by about 35 percent (Case Nos. 3 and 5 of TABLE 21). Case Nos. 6 and 7 present the effect of street sweeping efficiency on combined sewage overflow quality. The numerical difference in pollutant loadings between these two cases is about the same as that between Case Nos. 3 and 5.

Simulation analysis by others (47) using STORM indicates the amounts of

TABLE 21. POLLUTANT OVERFLOW AS AFFECTED BY STREET SWEEPING INTERVAL AND EFFICIENCY

Case No.	Street Sweeping		DWF	Annual Pollutant Overflow (lbs)	
	Interval (days)	Efficiency		SS	BOD
1	1	0.75	No	3193	603
2	3	0.75	No	10224	513
3	7	0.75	No	26883	4319
4	14	0.75	No	51769	3918
5	7	0.50	No	36243	5659
6	7	0.75	Yes	51337	24345
7	7	0.50	Yes	60889	25705

Note: Computation based on intercepting rate of peak DWF, zero storage and 12-year precipitation data.

1 lb = 0.453 kg

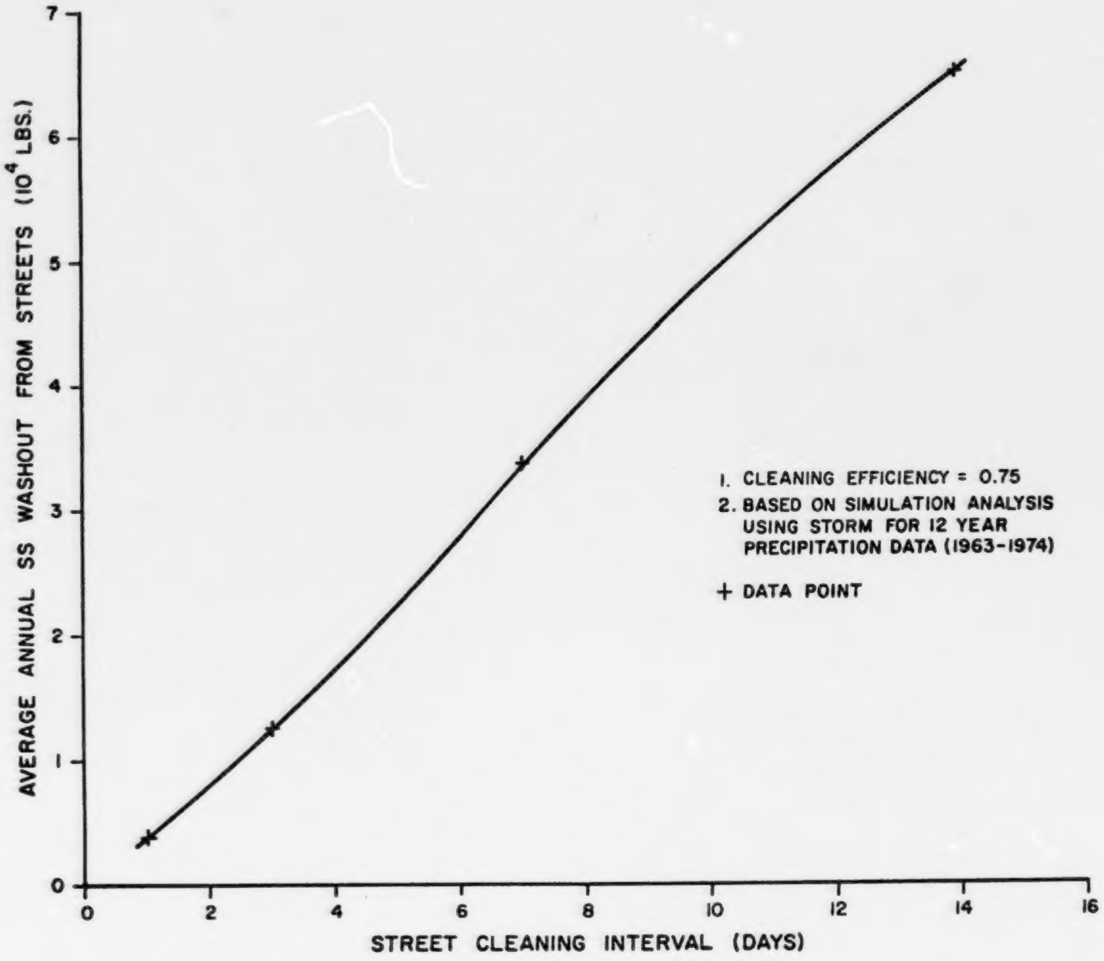


Figure 33. Effect of street cleaning interval on SS street washout

pollutant contained in urban runoff increases rapidly as the interval between successive street sweepings increase until the interval exceeds about 20 days. Beyond this interval, the amount of pollutants in the urban runoff becomes essentially constant, and the entire cost of street sweeping may be attributed to aesthetics. At lesser sweeping intervals, part of the cost may be attributed to reducing pollution.

SECTION XI

INTERCEPTORS FOR POLLUTION CONTROL

For effective combined sewage overflow pollution abatement, the "first flush", which usually occurs early in a storm runoff event, should be captured for treatment. The interceptor capacities required to contain the "first flush" for the alternative collection sewer systems considered in District A are defined herein. These interceptor capacities are further considered in Section XII to develop and compare alternative combined sewage overflow pollution control programs for the City of Elizabeth.

THE "FIRST FLUSH" PHENOMENA

As previously discussed, the outflow mass rate pollutograph in a combined system exhibits two peaks. The first peak results from the flushing of dry-weather flow solids deposited in the sewer and the second from street washout. The first peak is of low volume but with a high concentration and, perhaps, the larger mass of pollutants, while the second is of high volume but with much lower concentrations as well as less mass of pollutants. The outflow concentration pollutograph exhibits only one peak which occurs early in the storm when the flow rates are small. The peak concentrations may be an order of magnitude higher than that found in dry-weather flow. In considering the impact on a free-flowing stream, the peak of the overflow concentration pollutograph may be of primary concern. The characteristics of this peak are tabulated as follows:

Synthetic Storm Return Frequency	Peak Overflow Concentrate SS	Time to Peak Overflow Concentration	Runoff Volume* To Time of Peak Overflow Concentration		Maximum Flow Rate to Interceptor	
	(mg/l)	(minutes)	(mm)	(inches)	(m^3/sec)	(cfs)
5-year	20	56	2.50	0.098	7.05	249
	25	50	1.60	0.063	5.86	207
1-year	20	70	3.16	0.144	15.55	449
	22	60	1.40	0.055	4.33	153
1.3-month	92	60	0.30	0.012	0.99	35
	20	84	2.62	0.103	6.09	215

* Expressed in terms of area of District A (265 hectares or 655.1 acres)

A significant finding of the above tabulation is the need to control a greater volume of combined sewage for a 1.3-month synthetic storm than for a 5-year synthetic storm to achieve equal pollution abatement.

The estimated storage required to contain the critical first flushes from the entire City is less than 80,000 m^3 (21 million gallons), which is equivalent to about 4.6 mm (0.18 inches) over the drainage area. This storage capacity is being developed by the U.S. Army Corps of Engineers along the Elizabeth River near the Joint Meeting Treatment Plant.

Interceptor capacities required for containment of the first flush from District A are presented in TABLE 22 for storage basins of varying size. The location and operating mode of storage basins influence the amount of interceptor capacity required to limit the pollutants in overflows

TABLE 22. INTERCEPTOR CAPACITY REQUIRED TO CONTAIN FIRST FLUSH FROM DISTRICT A

Interceptor Capacity *** (cfs)

Storage* Capacity	Synthetic Storm Return Frequency 5-Year	1-Year	1.3-Month
0	249	549****	215
9.4	218	358	127
17.0	218	361	128**
41.3	20	137	49**

* expressed percent of runoff determined using SWMM and the synthetic, intermediate pattern, 5-year return period hyetograph. The storage basins are located as defined in Figure 22.

** with storage located at the confluence of the trunk and interceptor, the entire storm volume would be stored.

*** assumes intermediate pattern storms and suspended solids concentration of less than 20 mg/l in any overflow.

**** reduces to 153 cfs if the maximum allowable concentration in the overflow is 22 mg/l.

Note: 1 cfs = 0.0283 m^3/sec.

to a specified value. The operating mode has a greater effect upon storage located close to or at the confluence of the trunk and interceptor sewers than if located upstream. If storage is operated so that there is zero discharge from the basin until it is full, the interceptor capacities required for upstream and downstream storage locations for storms of various return intervals are tabulated below. This tabulation assumes intermediate pattern storms and a maximum permissible concentration of SS in the overflow of 20 mg/l using STORM default values.

Storage Location	Storage*	Interceptor Capacity — Synthetic Storm Return Frequency					
		5-year		1-year		1.3-month	
		m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
Upstream**	17	6.17	218	10.22	361	3.62	128
	41.3	0.57	20	3.88	137	1.39	49
Downstream	17	39.85	1407	15.52	548	No Discharge	
	41.3	25.20	890	0.28	108	No Discharge	

* In percent of runoff computed using synthetic, intermediate pattern, 5-year return interval storm hyetograph.

** Located where defined in Figure 22.

Runoff from the frequent storm (1.3-month return frequency) would be completely contained within the downstream storage volume. The rare storm (5-year return frequency) would require interceptor capacity equal to the peak runoff rate with 17 percent downstream storage volume and equal to about 63 percent of the peak runoff rate with 41.3 percent downstream storage volume. Less rare, but still severe, storms (1-year return frequency) would require about 52 percent greater capacity with 17 percent downstream storage volume, but about 21 percent less capacity with 41.3 percent downstream storage volume than those with upstream storage. This reversal in trend basically results from less dilution of the first flush with smaller amounts of storage. Where the interceptor capacities indicated above for downstream storage are in excess of those shown for upstrem storage, they could be reduced by changing the operating mode to permit discharges equal to the interceptor capacity during the storage fill period.

SECTION XII

ALTERNATIVE POLLUTION CONTROL PROGRAMS

While removing almost all pollutants from a 5-year synthetic storm might be desirable in areas draining to environmentally sensitive waters demanding very high quality effluent discharges, it is not normally required to protect the environment. In the development of a feasible program for acceptable pollution control in Elizabeth, a maximum treatment rate of 1.75 m^3/sec (40 mgd) (as is available to the City at the Joint Meeting Plant), and a storage capacity of about 80,000 m^3 (21 million gallons) near the treatment plant (as is being developed by the Corps of Engineers) are assumed to be an integral part of the cost-effective program.

The alternatives compared are shown in TABLE 23. The variables include (a) ratio of interceptor capacity to the peak 5-year synthetic storm runoff rate and (b) amount and location of in-system storage. The interceptor capacities investigated ranged from 1.4 to 100 percent of the peak 5-year storm runoff rate as computed from the intermediate pattern synthetic hyetograph. The overflow concentrations for the selected standard, intermediate pattern storms of 5-year, 1-year and 1.3-month return intervals were obtained using SWMM. The annual overflow frequencies and mass pollutant discharges were obtained using STORM and are based on analysis of a 12-year, continuous real rainfall record. The annual overflow characteristics are the primary influence in determining the "cost-effective" alternative. Hence, the findings are based on real rainfall events rather than on synthetic hydrologic phenomena.

For each alternative, interceptor sewers were selected and the cost estimated. The ratios of regulated flow to the peak 5-year synthetic storm runoff obtained from District A were used to modify the unregulated 5-year synthetic storm hydrographs of the remaining drainage districts to obtain the regulated outflow hydrographs required to size the interceptor sewer. Downstream storage of 80,000 m^3 (21 million gallons) is more than adequate for containment of the environmentally significant portion of runoff except for Case Nos. 6 and 7 where additional storage would be required. The cost of downstream storage included only the pumping station required to lift interceptor flow to the storage basins for later treatment. The unit cost of satellite and off-line storage of \$0.08 per liter (\$0.30 per gallon) was used in the comparison.

CONVENTIONAL COMBINED SYSTEM

Case Nos. 1 to 7 of TABLE 23 apply to conventional combined systems

TABLE 23. COMPARISON OF ALTERNATIVE COMBINED SEWER SYSTEMS

Case No.	Interceptor Capacity (cfs)	Ratio to Peak 5-Year Storm Runoff	In-System* Storage (%)	Combined Sewage Overflow Concentration (mg/l) 5-Year SS	5-Year BOD	1-Year SS	1-Year BOD	1.3-Month SS	1.3-Month BOD	Annual Overflow No. of Event	Annual Overflow SS (lbs)	Annual Overflow BOD (lbs)	Capital Cost ($10^6) Collection Sewer System	Interceptor	Pumping Station	Total Cost	Cost per Square Mile	$\frac{\Delta SS}{\Delta \$}$ (lbs/$10^6)
A	9	.006	0	1071	56	766.	55	33[illegible].	57	65.8	51337	24345	122.29	10.73	0	133.02	19.56	--
1	35	.025	0	400.	28.	250.	29.(156)	23.8	7.1(26)	40.3	21390	7140	122.29	15.20	1.80	139.29	20.48	4776
2	153	.109	0	35.7	7.9	21.4	6. (78)	28.6	8.4(18)	6.6	2700	620	122.29	32.10	7.30	161.69	23.78	1696
3	207	.147	0	25.	6.0	21.4	5.5(69)	26.7	6.8(4)	3.8	1150	260	122.29	35.9	9.60	167.79	24.68	1443
4	215	.153	0	23.1	5.5	21.3	5.2(58)	**	**	3.5	1000	230	122.29	36.5	10.30	169.09	24.86	1396
5	249	.177	0	20	4.3	21.4	4.6(44)	**	**	2.3	611	149	122.29	39.00	10.90	172.19	25.32	1295
6	549	.391	0	17	3.0	20	3.6(8)	**	**	0.2	23	9	122.29	51.70	16.80	190.79	28.06	888
7	1407	1.0	0	**	**	**	**	**	**	0.	0	0	122.29	97.57	23.90	243.76	35.85	464
8	20	.014	41.3(U)	19.3	6.9	22.3	6.3(154)	23.8	7.1(26)	1.8	995	630	126.81	13.70	1.05	141.56	20.82	5895
9	49	.035	41.3(U)	19.2	3.5	22.3	3.7(36)	**	**	1.1	268	192	126.81	19.50	2.49	148.80	21.88	3236
10	137	.098	41.3(U)	15.9	2.3	**	**	**	**	0.5	51	37	126.81	30.50	6.61	163.92	24.11	1660
11	128	.091	9.4(U)	29.	4.6	26.5	4.7(86)	**	**	2.7	859	242	123.16	29.50	6.24	158.90	23.37	1951
12	128	.091	17.0(U)	29.	4.6	26.9	4.7(36)	**	**	1.4	313	126	117.17	29.50	6.24	152.91	22.49	2565
13	218	.155	9.4(U)	20.	3.	25.9	4.2(56)	**	**	1.1	203	73	123.16	37.00	9.60	169.76	24.96	1392
14	218	.155	17.0(U)	20.	3.	26.2	4.2(18)	**	**	0.6	92	46	117.17	37.00	9.60	163.77	24.08	1667
15	105	.075	9.4(D)	70.	8.9	71.3	9.7(86)	**	**	3.3	1215	336	128.27	27.00	5.19	160.46	23.60	1827
16	105	.075	41.3(D)	21.7	3.	**	**	**	**	0.5	83	61	148.94	27.00	5.19	181.13	26.64	1065

* - percent of 5-year storm runoff volume.

U - upstream storage with locations defined in Figure 22.

D - one basin located at the confluence of the collection system and the interceptor.

** - no overflow

*** - number in parenthesis are overflow durations in minutes

Δ SS - reduction of SS overflow in lbs. with respect to SS overflow for Case A
Δ $ - incremental cost for the City in $10^6 with respect to cost for Case A
Note: CASE A assumes intercepting only peak dry weather flow.

1 cfs = 0.283 m^3/sec; 1 lb = 0.0453 kg; $1 per square mile = $0.00386 per hectare

with various rates of flow intercepted for treatment. The effect of increasing interceptor capacity on pollution control and costs are evident. Figure 34 presents for various ratios of interceptor capacity to the peak runoff rate of the adoped 5-year synthetic storm the estimated (a) combined sewage SS concentrations discharged to receiving waters, (b) annual number of overflow events from District A and (c) cost, assuming new sewers. The difference in cost between the systems results from the alternatives for interceptor sewer and pumping station capacity. The cost of the collection system is the same for all alternatives.

The findings in TABLE 23 are summarized as follows:

Case No.	Pollutants* Captured for Treatment (%)	Number of Overflows per year (1 hour or more)	Overflow in 1.3-month Storm Duration (Min.)
1	58	40.3	26
2	95	6.6	18
3	98	3.8	4
4	98	3.5	0
5	99	2.3	0
6	99.9	0.2	0
7	100	0	0

* % of annual SS overflow for Case No. A (TABLE 23) captured and treated.

In terms of pollution control, there appears to be little reason to provide facilities better than those included in Case Nos. 3 and 4. Based on the estimates developed herein, it appears the maximum justifiable cost for combined sewage pollution control is less than $96,000 per hectare ($25 million per square mile) and is about 21 percent greater than for Case No. 1 which would only capture for treatment about two-thirds of the pollutants. By utilizing the storage available in the combined system through devices to control routing as previously discussed, pollution control better than that offered by Case No. 4 might be achieved for somewhat more than $89,000 per hectare ($23 million per square mile) (Case No. 11 with a conventional system plus storage regulators).

ADVANCED COMBINED SYSTEM

An advanced system differs from a conventional system in that storage capacity and controlled routing is built into the system. The conventional combined system, designed to convey flows determined using the 5-year return frequency, synthetic, intermediate-pattern hyetograph, has sufficient volume to prevent overflow from storms with a return frequency of 1.3-month if flow routing devices are installed.

A storage facility may be operated to permit either (a) no diversion to storage until downstream sewer capacity is reached (to minimize operating cost), or (b) diversion to storage with controlled, low rate outflow until storage is full (to maximize pollution control benefits). With the first

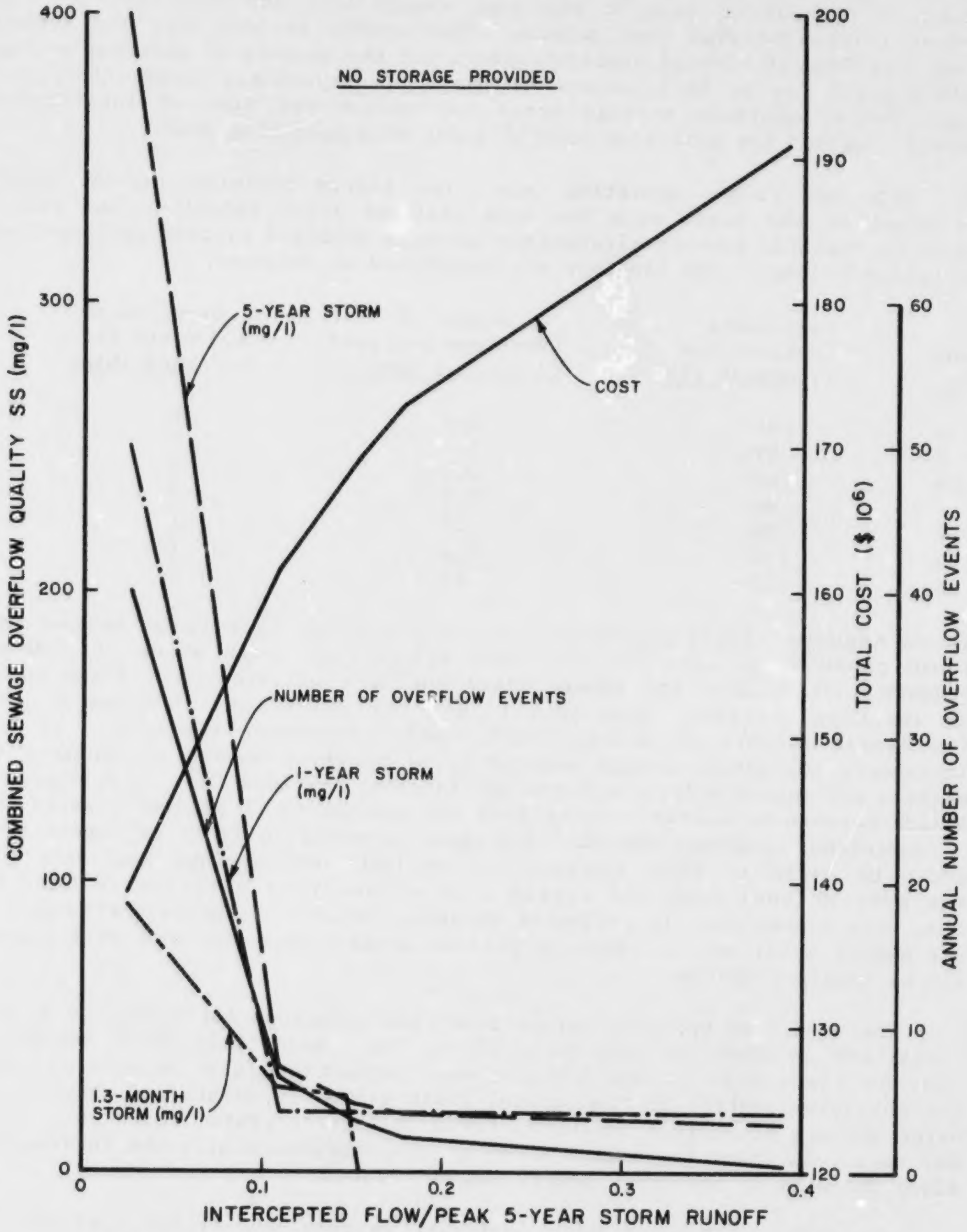


Figure 34. Pollution Control Costs - Conventional Systems

operating mode, the peak flow to be accepted for pollution control is about 15 percent or less of the peak runoff from the synthetic storm of 5-year return interval used herein. This amount is less than the reduced peak flow from the 5-year synthetic storm for the amounts of upstream storage investigated (up to 41.3 percent of the 5-year synthetic storm runoff volume). Hence, upstream storage would not reduce the size of intercepting sewers required for pollution control under this operating mode.

With the second operating mode, the highly polluted initial runoff is mixed in the basin with the less polluted later runoff. Case Nos. 8 to 14 of TABLE 23 compare alternative advanced combined systems with upstream in-system storage. The findings are summarized as follows:

Case No.	Pollutants Captured for Treatment (%)	Number of Overflows per year (1 hour or more)	Overflow in 1.3-month Storm Duration (Min.)
8	98	1.8	26
9	99+	1.1	0
10	100-	0.5	0
11	99+	2.7	0
12	99+	1.4	0
13	100-	1.1	0
14	100-	0.6	0

There appears little justification in providing facilities better than those provided in Case No. 8. This alternative costs about $81,000 per hectare ($21 million per square mile) but is predicated on a large volume of upstream storage. Case Nos. 11 and 12, and 13 and 14 compare costs for lesser amounts of storage with equal interceptor capacity. In both instances, increased storage reduces costs by about $3,900 per hectare ($1 million per square mile). All the alternatives provided for a high degree of solids capture at a cost ranging from six percent to 28 percent greater than a conventinal combined system. The upper permissible limit of expenditures would be fixed by that system which exploits the storage available with the conventional combined system most effectively (similar to Case No. 11). This system cost is estimated at about $89,000 per hectare ($23 million per square mile) and is about 19 percent greater than the cost of a conventional combined system.

The effect of upstream versus downsteam locations for satellite storage facilities is shown as Case Nos. 15 and 16. Relatively small amounts of storage (Case Nos. 11 and 15) are more advantageously located upstream for pollution control if the capital costs are to be maintained about equal. Large amounts of storage in the upstream collection system (Case Nos. 10 and 16) result in lower costs than those at the confluence with the interceptor, since the size of the trunk sewers can be reduced.

The relation between mass pollutant capture and system cost for the indicated range of interceptor capacity is presented in Figure 35.

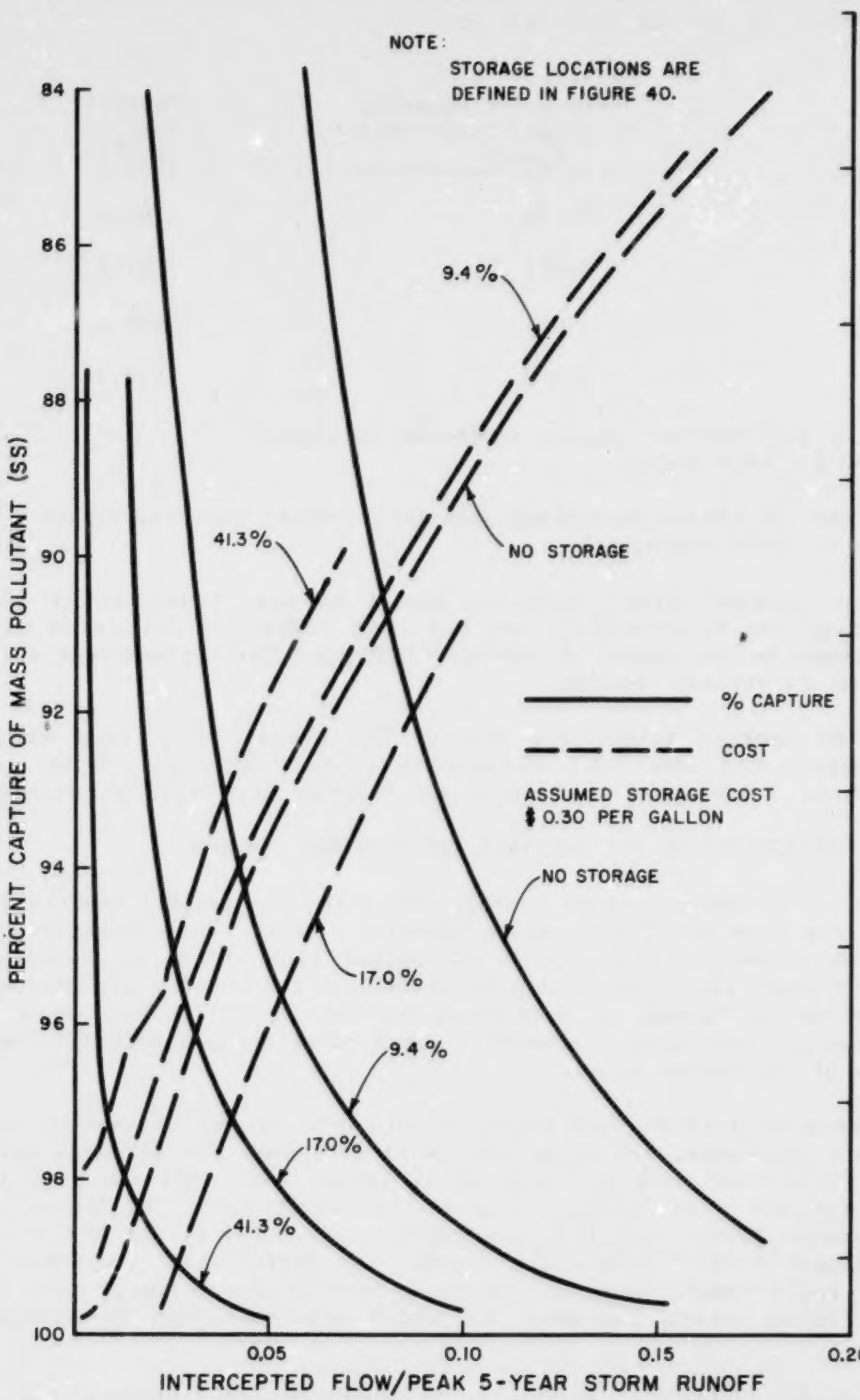


Figure 35. Pollution Control Costs - Advanced System

Interpretation of this figure indicates the following for a capture rate of 96 percent of the suspended solids:

In-System** Storage	Ratio: Interceptor Capacity to 5-year Peak* Storm Runoff	Capital Cost ($$10^6$)
0	0.124	164.4
9.4	0.057	151.2
17.0	0.031	136.0
41.3	0.006	138.5

* Based on intermediate pattern synthetic hyetograph adopted for this report.

** In percent of volume determined from intermediate pattern, 5-year synthetic storm hyetograph.

For lesser capture rates, there is almost no cost reduction if 41.3 percent storage can be provided, but the cost reduction increases with decreased capture as the amount of storage reduces. The minimum cost still occurs at about 17 percent storage.

For greater capture rates, the greater the amount of storage within the range tested, the lower the increase in costs. At about 98 or more percent capture, it becomes economical to provide 41.3 percent storage.

DESIGN STORM CONSIDERATIONS FOR SEPARATE AND COMBINED SYSTEMS

In design of a separate storm system, the pollution control aspects are less controlling than the flood damage aspects. Hence, cost-effectiveness is largely determined by benefits to be gained from relief of flooding. In design of a combined system, the pollutional control aspects are predominant and cost-effectiveness is determined by the facilities necessary to achieve appropriate pollution abatement. These conflicting standards affect the selection of the design storm.

As the sewered area contributing to a single outlet increases, the cost of sewers increases, and at a more rapid rate than the drainage area. This results since the relative lengths of larger size pipe required increases and the unit cost for pipe greater than about 2.1 m (84 inches) in diameter increases faster than the carrying capacity. Hence, for protection of a fixed property value from flood damage, the justifiable investment in storm drains would remain essentially fixed, and as the drainage area increases, the storm return frequency for which protection can be provided reduces.

In considering pollution abatement, however, in-pipe storage can be justified as a significant element of a cost-effective program. As previously discussed, the conventional combined system has about the same total pipe volume as the advanced combined system with 8.3 percent in-pipe storage, and can provide storage for about 3.8 mm (0.15 inches) of runoff with appropriate flow control devices (TABLE 10).

If a 2-year synthetic storm is used, the total volume in the sewer system would be reduced by about 5.1 mm (0.20 inches) or the potential for in-system storage would be significantly reduced. However, the cost of the collection system would be reduced by more than 20 percent, or more than $13,900 per hectare ($3.7 million per square mile). At a cost of $0.08 per liter ($0.30 per gallon), this saving could build more than 45,000 m^3 (12 million gallons) of storage or more than 13,600 m^3 of storage at $0.26 per liter (3.7 million gallons at $1.0 per gallon). These volumes are equivalent to more than 17.5 and 5.3 mm (0.69 and 0.21 inches) of storage respectively, or 37 percent and 12 percent of the runoff volume from the selected 5-year return interval storm hyetograph respectively. Hence, if sites are available for construction, satellite storage is more economical than using sewers sized for a larger return interval storm for pollution control. Designing sewers to provide flood control protection for less frequent return interval storms on the basis of pollution control is even less justified. The increased cost per hectare for a sewer system designed based on a 10-year return period storm is more than $17,400 per hectare ($4.5 million per square mile). The system would provide a potential storage volume for pollution control of about 7.1 mm (.28 inches) of runoff or about 18,500 m^3 (4.9 million gallons) of storage. Such design is not cost-effective for pollution control because of the high cost of such storage. Hence, selection of a 5-year return interval storm design criteria can be justified for pollution control if there is no other alternative for obtaining storage. If there are alternatives for providing storage, a 2-year design storm for flood control could be ample except in high risk areas where the potential for damage justifies greater protection and investment. With protection based on a design storm with a 2-year return interval, flooding of perhaps one-half hour duration would be expected every five years and lesser flooding at more frequent intervals.

SECTION XIII

APPENDICES

Page
A. ALTERNATIVE MODEL EVALUATION....127
  EQUATIONS OF MOTION....127
  COMPARISON WITH MANNING'S EQUATION....131
  COST OF SOLUTIONS....136
  FINDINGS....137
  MODEL OPERATING CHARACTERISTICS....137
  MODELS EVALUATED....138
    Corps of Engineers' STORM....139
    EPA - Storm Water Management Model (SWMM)....141
    University of Illinois - Storm Sewer Simulation Model....142
  COMPARATIVE EVALUATION....142
  THE RATIONAL METHOD....143
  METHODOLOGY....145
B. MODIFICATION OF SWMM AND STORM PROGRAMS....147
  SWMM....147
  STORM....148
C. QUANTITY AND QUALITY CONSIDERATIONS, SWMM VS STORM....150
  QUANTITY....150
  QUALITY....151
D. MODEL INPUT DATA....154
  LONG-TERM METEOROLOGICAL DATA....154
  FIVE-YEAR SYNTHETIC STORM HYETOGRAPH....159
  HYETOGRAPHS OF LESS INTENSE STORMS....161
  DEFINITION OF DISCRETE ELEMENTS....165
  HYDRAULIC PROPERTIES AND LAND USE....172
  DISTRICT A DATA FOR STORM....172
  AREA AND LAND USE FOR OTHER DISTRICTS....184
  DRY-WEATHER FLOW QUANTITY AND QUALITY....185
  UNIT COST DATA....192
    Sewer Costs....192
    Storage Costs....192
    Pumping Station Costs....195
    Treatment Costs....195
E. CALIBRATION OF STORM RUNOFF COEFFICIENTS....197

APPENDIX A

ALTERNATIVE MODEL EVALUATION

There are at least 24 models available for the assessment, planning, design and control of sewer systems (20). These models compute time-varying runoff as opposed to steady state analysis. However, they differ in scope and purpose and in the mathematical detail of routing flow through sewers. In essence, the difference is in the approximations made to model the complete momentum equation. The simplest models use Manning's equation or similar formulae. Simplification of the momentum equation affects the sizing of collection sewers and downstream interceptor, storage and treatment facilities (17). As part of the study, Resource Analysis, Inc. (RAI), Cambridge, Massachusetts made an evaluation of sewer design equations. In addition, RAI evaluated the applicability of available stormwater management models.

Stormwater simulation models use the governing equations of motion - continuity and momentum - to numerically predict the flow in stormwater systems. Following is an analysis of the momentum equation, evaluating the relative importance of the various terms in this equation. A qualitative analysis of the effect of eliminating some of these terms was investigated, focusing on the accuracy of the simulation predictions, the cost of unproved accuracy, and the potential mis-design of the resulting structure.

EQUATIONS OF MOTION

The governing equations of motion for one-dimensional unsteady open channel flow (21,22) are:

Continuity Equation:

$$\underset{(a)}{A\frac{\partial v}{\partial x}} + \underset{(b)}{v\frac{\partial A}{\partial x}} + \underset{(c)}{B\frac{\partial y}{\partial t}} = \underset{(d)}{q} \qquad \text{(A-1)}$$

Momentum Equation:

$$g(\underset{(e)}{S_0} - \underset{(f)}{S_f}) - \underset{(g)}{g\frac{\partial y}{\partial x}} - \underset{(h)}{v\frac{\partial v}{\partial x}} - \underset{(i)}{\frac{\partial v}{\partial t}} - \underset{(j)}{\frac{vq}{A}} = 0 \qquad \text{(A-2)}$$

Where:

A = the cross-sectional area of the channel,

v = the mean flow velocity,

x = the downstream distance along the channel,

B = the surface width of the channel,

S_f = the friction slope,

S_o = the bed slope,

y = the water depth,

t = the time,

g = the gravitational constant, and

q = the lateral inflow.

Figure A-1 presents a definition sketch for the equations of motion.

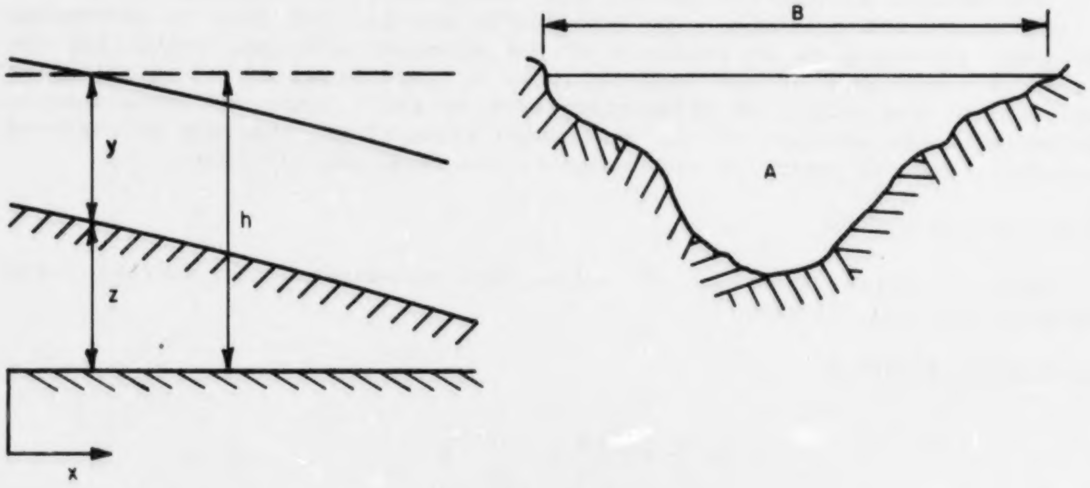


Figure A-1. Definition sketch for equations of motion

The assumptions made to arrive at these equations are:

1. The flow is one dimensional, i.e. uniform velocity distribution

across each cross section, and the free surface is horizontal across the section,

2. pressure is hydrostatic, i.e., vertical acceleration is neglected, and

3. boundary friction and/or turbulence may be accounted for through Manning's equation.

These equations can also be applied to cases of flow in partially full pipes, with the proper definition of area and friction terms. The terms in Equations A-1 and A-2 can be described as:

(a) prism storage term due to variations in velocity with space,

(b) wedge storage term due to variations in cross-sectional area with space,

(c) rate-of-rise term that describes the changes in storage due to water surface elevation variations over time,

(d) lateral inflow term that represents the net inflow from lateral sources,

(e) bottom slope term,

(f) friction slope term,

(g) water surface slope term (pressure force),

(h) convective acceleration due to spatial variation in velocity,

(i) local acceleration due to temporal variation in velocity, and

(j) term indicating momentum change due to lateral inflow.

Various mathematical models for flow routing have been proposed in the literature. These models can be classified according to which terms of the governing equations they include in their solution. Some general classifications are (neglecting lateral inflow):

1. Storage Routing Models - use continuity equation only,

2. Kinematic Wave Models - use continuity equation, and friction (f) and bottom slope (e) terms of momentum equation,

3. Diffusion Wave Models - also include pressure term (g) of momentum equation,

4. Dynamic Wave models - use all terms in momentum equation.

Only the Dynamic Wave Models can describe exactly all of the effects present in a flow motion since these are the only models that retain all of the terms in the momentum equation. For many practical situations, some of these terms may be ignored resulting in a significant reduction in the cost of simulation runs. An order of magnitude analysis of the terms in the momentum equation can give some indication as to the importance of each of these terms. A compilation of the range of typical values for these terms is presented in TABLE A-1. These values were obtained from Schaake (23), Henderson (21), Eagleson (24) and experience with these models.

TABLE A-1. RANGE OF TYPICAL VALUES FOR EACH TERM IN THE MOMENTUM EQUATION

Term		Overland Flow (ft/sec^2)	Pipe & Gutter Flow (ft/sec^2)	Streamflow (ft/sec^2)
gS_o	(e)	0 - 3.20	0 - 1.60	0 - 0.30
gS_f	(f)	0 - 3.20	0 - 1.60	0 - 0.30
$g\frac{\partial y}{\partial x}$	(g)	0 - 0.10	0 - 0.60	0 - 0.005
$v\frac{\partial v}{\partial x}$	(h)	0 - 0.01	0 - 0.30	0 - 0.002
$\frac{\partial v}{\partial t}$	(i)	0 - 0.01	0 - 0.30	0 - 0.0005
$\frac{vq}{A}$	(j)	0 - 0.001	0 - 0.05	0 - 0.005

Note: $1\ ft/sec^2 = 0.305\ m/sec^2$

The local acceleration (i) and lateral inflow (j) terms appear to be relatively small for all cases. For overland flow and steep streams, the water surface slope (g) and convective acceleration (h) terms also appear to be of small magnitude when compared to the bottom (e) and friction (f) slope terms. Henderson (21) has shown that for most streams and for Froude numbers less than 1, the acceleration terms due to spatial (h) and temporal variations (i) in velocity are much smaller than the acceleration due to the water surface slopes (g). For Froude numbers higher than 1 all three terms (g), (h) and (i) are of the same order of magnitude but these cases only occur in mountain torrents where the bottom slopes (e) are much larger. Flows with high Froude numbers may also occur in urban storm drainage systems. But in these systems, the effects of control structures, e.g., weirs, flow dividers, etc., produce more noticeable flow changes than the additional terms (g), (h) and (i) in the momentum equation.

For flow in pipes, the water surface slope (g) and convective accelea-tion (h) terms may be large enough to be significant relative to the bottom (e) and friction (f) slope terms. Terms (g) and (h) are the key to modeling the dynamic (e.g., backwater, subsidence) effects that are sometimes present in pipe flows. The major difference between a pipe system and stream system is that pipe systems are usually surrounded by highly urbanized catchments. These catchments respond much more quickly and with higher lateral inflow rates than non-urban catchments. Therefore, terms (g) through (j) are relatively larger for pipes than for streams.

An order of magnitude analysis can only give an indication as to the relative importance of the different terms. But it is unlikely that a very small term in the momentum equation could have any significant effect on the solution to the equations of motion. With this condition, only the bottom (e) and friction (f) slopes appear significant for overland flow and steep sloped streams. For mild sloped streams and for pipe flows, the water surface slope (g) may be significant. In addition, for pipes the convective acceleration term (h) may also be significant.

COMPARISON WITH MANNING'S EQUATION

If Manning's equation is used as the basis to establish a relationship between depth of flow and discharge, the influence of the smaller terms in the momentum equation is of interest. Manning's equation provides a relationship among the flow velocity, v, depth of flow and shape of the channel cross section as expressed through the hydraulic radius, R, and the friction or energy grade line slope, S_f. The relationship is:

$$v = \frac{1.49}{n} R^{2/3} S_f^{\frac{1}{2}} \qquad \text{(A-3)}$$

In which "n" is Manning's resistance coefficient. Equation A-3 may be solved for the friction slope, S_f, to yield:

$$S_f = \frac{v^2 n^2}{(1.49)^2 R^{4/3}} \qquad \text{(A-4)}$$

which may then be substituted into the complete momentum Equation A-2. The latter is solved for the flow velocity, v, giving:

$$v = \frac{1.49}{n} R^{2/3} \left(S_0 - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} - \frac{1}{g}\frac{\partial v}{\partial t} - \frac{vq}{gA}\right)^{\frac{1}{2}} \qquad \text{(A-5)}$$

If the flow is steady, $(\frac{\partial v}{\partial t} = 0)$, uniform, $(\frac{\partial v}{\partial x} = 0)$, and there is no lateral inflow, (q = 0), all but the first of the terms in parentheses disappears and Equation A-5 is reduced to the familiar form of Manning's equation that is:

$$v = \frac{1.49}{n} R^{2/3} S_0^{\frac{1}{2}} \qquad \text{(A-6)}$$

The significant difference between Equations A-3 and A-6 must be noted. In Equation A-6 the term S_o, is a known geometric property of the channel, hence a unique, known relation exists between velocity and hydraulic radius (or, as more commonly expressed, between stage and discharge). But, in Equation A-3 the slope, S_f, is in general not known, and, from Equation A-5, the friction slope depends on the rates at which the depth and velocity are changing.

Equations A-5 and A-6 may both be considered to be forms of Manning's equation. However, the latter is a special case and thus limited in its application. The effect of the extra terms in Equation A-5 is readily demonstrated for the case of a prismatic channel of constant bottom slope and having no lateral inflow. For this case, the two equations provide relations between stage and discharge which may be plotted in Figure A-2.

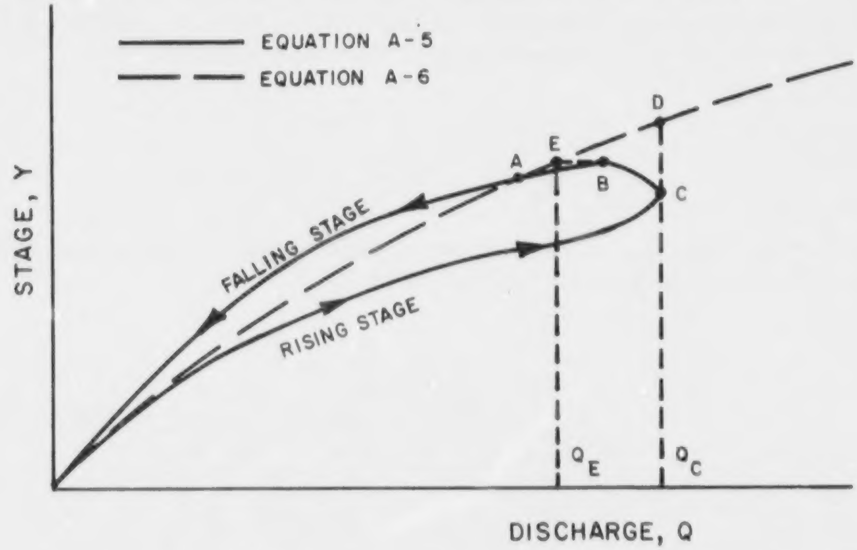


Figure A-2. Uniform flow rating curve and loop rating curve

Equation A-6 gives a single-valued relation, while Equation A-5 is multivalued according to whether the stage is rising or falling with time. The latter describes a loop-type rating curve.

The relation between stage and discharge at a given channel section depends on whether one is on the rising or falling limb of the hydrograph. Although not shown in Figure A-2, the shape the loop rating curve is not unique, but is directly dependent on the shape of the inflow hydrograph.

Three points on the loop rating curve, A, B, and C, are of particular interest. At point A, the instantaneous gradient of depth with distance is identically zero and Manning's equation in the form of Equation A-6 is applicable. However, neither the peak discharge nor the peak stage occur at this point. The peak discharge occurs at point C prior to the occurrence of the peak stage which is at point B. Thus, the peak stage and discharge develop a phase shift which makes it difficult to determine the peak stage from the peak discharge.

If by routing an accurate value for the peak discharge, Q_C, is determined, the peak stage at point B must still be determined. Substitution of the peak discharge, Q_C, into Manning's Equation A-6 would yield a stage at point D which is too high. In this sense, use of Manning's equation will in general lead to conservative predictions of peak stage.

This tendency to overestimate the peak stage is reinforced since the peak discharge and peak stage do tend to subside with distance down the channel. This effect would not be predicted without the higher order terms of the momentum equation. Hence, any procedure such as direct translation of the inflow hydrograph to downstream locations or the kinematic routing model, both of which fail to reduce the peak discharge, will overestimate the discharge and thus add to overestimation of the peak stage.

The problem may also be reversed. Substitution of the peak stage, Y_B, into Equation A-6 will yield a discharge, Q_E, which is clearly smaller than the true peak, Q_C. Hence, Manning's equation can tend to underestimate the peak discharge.

A similar situation occurs when one uses the Manning's equation to estimate the discharge capacity of a structure. The geometric properties of the structure together with the given bed slope, S_o, can be substituted into Equation A-6 to yield the capacity of the structure. However, the friction slope, S_f, observed in the neighborhood of such control is frequently less than the bed slope, S_o, and the true discharge capacity as yielded by Equation A-3 will be less than that obtained using the bed slope directly. In this sense, use (or misuse) of the Manning's equation can lead to overprediction of the discharge capacity of a structure.

In summary, the use of Manning's equation, without the other terms in the momentum equation, is deficient in at least the following respects:

1. It fails to produce the loop-type rating relation.

2. It fails to produce a reduction of the peak discharge.

The magnitude and importance of these effects depend on the particular problem under study and may often be negligibly small. In some classes of problems, the need to include these effects is obvious. For example, in long natural channels with extensive flood plain storage or in river systems which include ponding areas and reservoirs, these effects may be very pronounced. On the other hand, urban drainage systems typically involve short, steep channels with well defined boundaries; in these cases the indicated effects may be secondary and are often negligible.

Simple criteria for determining when these effects may reasonably be neglected are not readily available. Until these criteria are forthcoming, considerable reliance will continue to be placed on the judgment and experience of the design engineer. Hopefully, the existence of computer models which allow retention of all terms in the momentum equation will permit the development of sounder judgment and the eventual preparation of precise criteria.

Other factors affect the accuracy of results from simulation models. For example, inaccuracies in rainfall measurements and/or the simplifications used to model a prototype system within a limited set of model elements (i.e., catchments, pipes, flow dividers, etc.) may introduce significant errors. In fact, these errors will in general be much larger than those resulting from neglecting some terms in the momentum equation. It is usually more important to model the effects of a flow diversion structure accurately than to model the subsidence of a flood wave in the input pipe exactly.

The preceding conclusions apply only to the case of open-channel flow in which downstream conditions do not have a significant effect on the upstream stages. This idealized situation frequently is not applicable. If a channel is terminated by a control structure such as a weir, constriction, culvert, etc., with discharge characteristics such that the stage in the channel immediately upstream of the structure must build up to a level greater than normal, a backwater profile will form for some distance upstream where the stage will be above normal. The actual values of stage will exceed those calculated by a simple application of Manning's equation with $S_f = S_o$. Manning's equation will then yield stage values which are too low, rather than too high as in the preceding discussion. The same type of behavior can arise when anlyzing flows in sewer systems. When the flow rates and/or downstream boundary conditions are such that the sewer pipes flow full and surcharinging occurs, the relationships defined by the normal flow condition, $S_f = S_o$, are no longer applicable. The friction slope, S_f, in these cases is related to differences in water level in the surcharged sewers which, as in the backwater example, will be higher than the levels given by Manning's equation with $S_f = S_o$. The difficulty lies not with Manning's equation per se. Rather, the difficulty stems from the inappropriate use of the assumption that the channel (or pipe) slope and the friction slope are equal, i.e. $S_f = S_o$. Manning's equation is valid where written in terms of an appropriate friction slope. However, the friction slope depends on the flow conditions. Thus, in general, the true friction slope is a dependent variable of the problem and is not known in advance. The assumption of $S_f = S_o$ can be made only when the validity of the assumption is known. If this assumption is indiscriminately built into a sewer design program (for example), considerable caution should be used in interpreting the model's results, especially if the results are to include predictions of stage or piezometric grade line elevation.

Figure A-3 presents a surcharged simple pipe system consisting of three links and manholes at each of the two nodes.

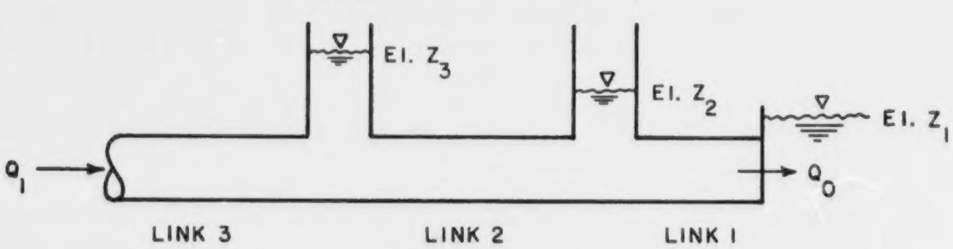


Figure A-3. Surcharged simple pipe system

The inflow to the system is a discharge from upstream sources and is designated as Q_1. The outflow, Q_o, is into a reservoir whose free surface is at elevation Z_1. The instantaneous water levels in the manholes are at elevations Z_2 and Z_3 as indicated. If the reservoir elevation, Z_1, is known as a function of time (e.g., constant, as in the case of a large reservoir), a mathematical relationship can be derived which expresses the manhole water surface elevations as a function of the inflow, Q_1. A particularly simple form of the relationship is obtained if a linear relation is assumed between discharge and head and the inertial effects of the water columns can be neglected. In this simplest representation the relationship between Q_1 and Z_3 takes the form:

$$a_1 \frac{d^2 Z_3}{dt^2} + a_2 \frac{dZ_3}{dt} + a_3 Z_3 = f\left[Q_1, \frac{dQ_1}{dt}, Z_1, \frac{dZ_1}{dt}\right] \qquad \text{(A-7)}$$

in which the coefficients, a_1, a_2 and a_3, depend on the geometry and hydraulic resistance of the system. The left-hand side of the equation is analogous to a simple spring-mass-dashpot system. The right-hand side is a function of the boundary conditions which serve as a forcing function.

One property of the solution of this equation is a phase shift between the forcing function and the response, Z_3. Among other things this implies that in general the peak inflow, $(Q_1)_{max.}$, and the peak water level in the manhole, $(Z_3)_{max.}$, do not occur at the same time. Thus, it is not possible to relate the peak flow and stage (i.e., manhole water level) in a simple manner. This behavior is analogous to that attributed earlier to the effects of channel storage in the open channel case, and again points out the problem of any routing procedure which fails to take these storage effects into account.

COST OF SOLUTIONS

Cost reductions for simulation runs may be attained by neglecting terms from the momentum equation. These reductions in cost are indirectly a result of solving a simpler set of equations. Such an action would allow the models to:

1. use larger Δt and Δx grids in the finite different formulations or
2. step-down to a less numerically complex solution of the equations (e.g., from the method of characteristics to a finite difference scheme).

Any of these two steps would result in significant cost reductions in simulation models.

From a qualitative point of view, some estimates of the effect of each term can be made. Schaake (23) has shown that neglecting the temporal variation in velocity term (i) can reduce the cost of simulation runs by at least a factor of 2. This is a result of the larger time steps that can be used for the finite difference schemes when $\Delta v/\Delta t$ is not in the equations. A similar reduction in cost can be obtained when the spatial variation in velocity term (h) is eliminated. In this case larger Δx steps can be used. Neglecting the water surface slope term (g) may not have as large an effect on the cost of solutions as the previous two terms. But this term can cause stability problems. In some cases, neglecting it may allow for much larger Δt and Δx grids and therefore result in significant cost reductions. The final term (j), the acceleration imparted on the lateral inflow, would have no significant effect on the cost of simulations since it only involves one additional operation.

In summary, if the variation in velocity terms (h) and (i) could be neglected, significant reductions in simulation costs could be obtained.

No detailed comparison of cost reductions versus number of terms modeled for the momentum equation has ever been published. Most evaluation and comparison studies have focused on overall differences between models and not on a comparison of the details of the solutions to the basic equations. Until such a study, which should involve extensive experimentation, is carried out, only qualitative statements can be made.

FINDINGS

1. In view of the relative magnitude of the different terms (TABLE A-1) and the increase in cost of solutions when additional terms are retained, overland flow can be accurately modeled with only the bottom slope (e) and friction slope (f) terms.

2. Modeling of steep streams can also be adequately modeled with only these two slope terms.

3. For streams where stage values are important, and for long reaches of mild slope streams, the water surface slope (g) and convective acceleration (h) terms need to be retained. These terms will introduce the dynamic effects which may exist in these cases. Simulation of pipes in urban systems may theoretically also require that these two terms be retained. However, other urban effects and modeling decisions are normally more significant than these terms.

4. Models which include more than the bottom and friction slope terms increase the cost of simulation runs. These "full equation" models should be used only where necessary, such as the case where stage values are important and where off-channel storage effects are significant.

5. Other factors, such as inaccurate rainfall data, or inaccurate modeling of control structures, are probably more important than the small errors introduced by using a simple model such as the Kinematic Wave Model.

6. In cases where backwater effects or surcharging of pipes is present, it is inappropriate to use the assumption that the channel (or pipe) slope and the friction slope are equal, i.e., $S_f = S_o$. Manning's equation can only be used in these cases if the appropriate friction slope is computed. But in these cases the friction slope is a dependent variable and is not known in advance.

MODEL OPERATING CHARACTERISTICS

The desired operating characteristics of the chosen simulation model (or models) are:

1. Ability to simulate

    (a) the surface runoff quantity,

(b) the operation of the trunk sewers, and

(c) any storage (in-pipe or satellite).

2. Ability to run on continuous records to check the overflow characteristics of the system over a period of record.

3. Ability to simulate quantity based on physical parameters.

4. Capability to be transferable from one area to another within a city and its environs without extensive calibration exercises.

5. Ability to predict the quality of surface runoff during wet-weather conditions.

6. Capability of handling the sanitary and dry-weather quality aspects of the system.

7. Ability to handle the pollutant transport in the trunk system on a conservative (i.e., no time decay) basis due to the small size of the drainage systems and hence short in-system time.

8. Ability to simulate directly the quality of overflows.

In the above synopsis "quality" has been used in the generic sense. For urban purposes it appears that the following constituents are of importance.

a) Biochemical oxygen demand (BOD)
b) Chemical oxygen demand (COD)
c) Suspended solids (SS)
d) Settleable solids
e) Nitrogen/nitrates
f) Phosphates
g) Grease
h) Coliforms
i) Heavy metals

The ability to simulate the pick-up, transport, etc. of the above pollutants is relatively crude at present. In particular, little or no capability to simulate heavy metal pollutants exists. However, the models discussed below are evaluated on their ability to handle pollutants in terms of the "ideal" set presented here.

MODELS EVALUATED

Following an initial survey, attention was focused on three models for a detailed review. These models were:

- Corps of Engineers Storage, Treatment, Overflow, Runoff Model (STORM)

- Environmental Protection Agency Storm Water Management Model (SWMM)
- University of Illinois Storm Sewer Simulation Model

The thorough analysis of urban models by Brandstetter (25) has been used as a general reference in much of the discussion on models which follows. Recently, the same author prepared a more comprehensive comparison of urban runoff models (26). The following sections review each of the above models, and recommend that both STORM and SWMM be used to provide adequate simulation capability.

Corps of Engineers' STORM

STORM was developed in early 1974 by Water Resources Engineers, Inc. (WRE) of Walnut Creek, California under contract to the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers. The program is an outgrowth of work performed by WRE for the Environmental Protection Agency and the City of San Francisco. The following comments on the program are based primarily on the User's Manual (27) and discussion with WRE and HEC personnel. The program is intended primarily for the evaluation of stormwater storage and treatment capacity required to reduce overflows to the receiving waters to acceptable (defined externally) levels. The model recognizes that the intense short-duration storms, so often used as "design" storms, may well be completely contained through natural and artificial storage mechanisms so that no untreated overflows occur. Alternatively, a series of moderately sized storms may load the system to the point where untreated releases occur. The program is thus designed to account for the characteristics of the area. Lumped storage and treatment capacities are also considered. The model considers the interaction of eight variables in determining the operation of the system:

1. Precipitation
2. Air temperature (for snowmelt computations)
3. Runoff
4. Pollutant accumulation on the land surface
5. Land surface erosion
6. Treatment rate
7. Storage
8. Overflows from the storage/treatment systems

The model operates on land uses, including single family residential, multiple family residential, commercial, industrial, parks and undeveloped areas. The program is designed to simulate hourly stormwater runoff and quality for a single catchment. It is a continuous simulation model with the capability of simulating a number of years of record but can also be used for selected single events. The model, as prepared by WRE, does not simulate dry-weather flow and its associated quality components. Modifications to include dry-weather flow simulation and up to 20 land uses have been made in the latest version of STORM (28).

The runoff from the catchment is computed using runoff coefficients and hourly precipitation data for a single rain gauge in the basin. Depression storage is simulated as the prime mechanism of reducing rainfall to effective rainfall. This depression storage is allowed to recover at a constant rate to account for evapotranspiration. The runoff from the area is computed by applying a simple runoff coefficient to the effective rainfall. This coefficient is a weighted average of the coefficients for pervious and impervious areas and does represent losses due to infiltration. A similar coefficient is used to simulate runoff during periods of no precipitation. Runoff from snowmelt is computed using a degree-day technique.

Six water quality constituents are computed for different land uses. These constituents are:

1. Suspended solids
2. Settleable solids
3. Biochemical oxygen demand (BOD)
4. Nitrogen (N)
5. Orthophosphates (PO_4)
6. Coliforms

These quality components are computed from non-linear functions considering the daily rate of dust and dirt accumulation, the percent of each pollutant contained in the dust and dirt, street sweeping practices and days between runoff events. Erosion is computed using the universal soil loss equation. The BOD, N and PO_4 runoff rates depend on the pickup rates of the suspended and settleable solids.

The model is very much a "black-box" type model. It does not route the runoff quantity or quality in a sewer or channel network. The routing aspects have to be reflected in the runoff coefficients. The computations of the treatment, storage and overflow processes at the single outflow from the system are performed by volume and pollutant mass balance only. A very simple logic is employed as follows: if the hourly runoff volume exceeds the treatment capacity, the excess runoff is diverted to storage; once the storage capacity is exceeded, the excess runoff becomes untreated overflow. During dry-weather conditions, the treatment capacity is used to draw down the volume in the storage element. The quality of the runoff is not modified in storage. Simple plug flow routing is used. Similarly, the simulation of the treatment facility does not include the quality improvement to be expected.

The model is completely based on the use of empirical coefficients for both stormwater runoff and quality computations. These coefficients were provided by the user for the runoff case, and are internal to the program in the quality aspects. Verification of the various equations appears to be limited to date, and the dependence of the runoff and quality coefficients on the catchment characteristics does not appear to have been fully established.

The model is useful primarily as a planning tool to estimate the approximate number and magnitude of overflows for various combinations of storage and treatment. It is a useful screening tool in these aspects.

EPA - Storm Water Management Model (SWMM) (6,29)

This is probably the most comprehensive mathematical model for the simulation of storm and combined sewer systems. It computes runoff quantity and quality from the watershed by using a number of overland flow elements to simulate the initial collection process, and a converging branch sewer network to simulate the trunk sewer system. It can also handle up to three storage basins and one treatment plant, together with the appropriate flow diversion structures.

The model, as initially developed and used in this work, was primarily an event simulator - its complexity and cost effectively limiting its use to a single storm at a time. However, the latest version has incorporated continuous simulation capability. Spatial rainfall variations over a basin can be taken into effect since independent inputs can be applied to each subcatchment. The runoff from the overland flow areas is simulated using the kinematic wave equations. Depression storage is accounted for, and infiltration from pervious areas is computed using Horton's equation.

The stormwater quality is computed using non-linear functions which are a function of land uses, street cleaning practices, pollutant accumulation and rate of runoff. The pollutants considered are of a wider range than considered in STORM and include suspended and settleable solids, BOD, COD, coliform bacteria, nitrogen, orthophosphates, oil and grease. Daily and diurnal variations in the dry-weather flow and quality are accounted for and are estimated from land use characteristics or directly from input data.

The flow in the sewer network, as included initially in the model and used in this work, was simulated using the kinematic wave equations. Complete continuity and momentum equations are included in the current model version (30). The elements in the network can include manholes and conduits from the selection of 12 internally stored geometric shapes, as well as three additional arbitrary shapes. Other elements in the network are diversion structures and up to two satellite storage facilities. Pumping stations with constant pumping rates can be included. Surcharging can be handled by assuming all flow in excess of full pipe flow is stored in the next upstream manhole. The model also allows the option of letting the program size the sewer to eliminate such surcharging.

Quality routing through the trunk sewer system is primarily based on an approach of pure mixing in each pipe element. Decay is allowed for the BOD, and scour and deposition of suspended solids is simulated. Other quality reactions and interactions are not considered.

Unlike STORM, SWMM can simulate the operation of up to nine unit treatment processes which can be inserted in series for a single overflow treatment facility. It thus allows more accurate estimation of the pollutant mass loads on the receiving waters.

The model also has the ability to simulate both quantity and quality behavior in the receiving water system.

The model has been applied to a large number of watersheds. In general, it has been found that the accuracy of the runoff quantity computations for basins ranging from 4-2000 hectares (10-5000 acres) was relatively good. If adequate segmentation rules are applied, the model can be successfully used for runoff prediction in most urban areas. On the other hand, as in similar models, significant doubts still exist as to the adequacy of the runoff quality prediction mechanisms. It is not yet known how generally applicable are the regression equations relating runoff quality to land use. Although they can be calibrated for individual basins, they can not yet be used with complete confidence for prediction purposes.

University of Illinois - Storm Sewer System Simulation Model (7)

This model basically computes nonsteady flows in a converging sewerage network using a solution of the dynamic wave equations. The routing is based on a first-order explicit finite difference solution to the characteristic equations of the dynamic wave equation. It can consequently consider upstream and downstream flow controls, backwater effects and flow reversals. It does not, however, include surcharging and pressure flows. A certain number of controls, diversions and pumps can be included but only at the outlet from the system. The model does not include any quality effects. Similarly, overland runoff from precipitation is not computed. Rather, the required flows must be entered as separate hydrographs. Dry-weather flows must be handled in a similar fashion.

The model includes a comprehensive formulation of open channel flow routing in circular sewers. It includes a feature for the sizing of circular pipes to accommodate peak flows. However, its lack of overland runoff and quality simulation capabilities limits its usefulness for the present study. In a recent study by Chow and Yen (31) for EPA, these capabilities have been incorporated into the model.

COMPARATIVE EVALUATION

The work under this project mandated use of a model or models which would be capable of being used in:

a) Planning of required facility locations and sizes, and

b) Evaluating alternative designs.

In addition, the models must be readily transferable from basin to basin within the urban area.

In light of the above discussion it appeared:

1. The University of Illinois model was of no advantage.

2. STORM included the storage/treatment capabilities required and, in particular, allowed comparison of a wider range of treatment/storage alternatives than could be economically considered using SWMM. It suffers from calibration problems.

3. SWMM was the most suitable model for the design aspects of the current study. Its RUNOFF and TRANSPORT Blocks have the required capability to simulate the stormwater quantities involved. Additionally, it considered a wide range of pollutants. However, the pollutant pick-up simulation mechanism must still be regarded as suspect and subject to calibration.

THE RATIONAL METHOD (32)

This method has been widely used for the determination of peak rates of stormwater runoff. It relates peak runoff rate to rainfall intensity by the formula,

$$Q = Cia \qquad \text{(A-8)}$$

where Q, the peak flow, is dependent upon rainfall intensity, i, and a runoff coefficient C which attempts to weight all the variables affecting runoff rates, and the tributary area, a. When applying the Rational Method the intensity is the average rainfall intensity for a storm having a duration equal to the longest time of travel in the system being designed (overland + pipe) and the runoff coefficient is a composite of published coefficients as weighted by land use characteristics. In this study, the maximum and minimum values of the composite runoff coefficient were obtained using the runoff coefficients published on page 51 of Reference 32.

To compare runoff rates determined by the Rational Method with those using SWMM, flows, developed using SWMM Model and the synthetic, intermediate pattern, hyetograph shown in Figure D-5, were used in the Rational Method formula. A corresponding runoff coefficient was obtained using the rainfall intensity determined from the appropriate intensity-duration frequency curve with the estimated time of concentration. The time of concentration includes the suggested inlet time of five minutes (32) and time of travel in trunk sewers, which is estimated from SWMM output assuming pipes are flowing full. TABLE A-2 summarizes the SWMM "C" values calculated at various points in District A and compares the peak 5-year synthetic storm flows obtained using the Rational Method with peak flows using SWMM. The Rational Method results in runoff rates from 30 to 60 percent lower than runoff rates determined using SWMM.

TABLE A-2 PEAK FLOW COMPARISON BETWEEN RATIONAL METHOD AND SWMM

Area (acres)	SWMM Flow (cfs)	Intensity (in/hr)	SWMM "C"	Rational "C" Range	Time of Concentration (Minutes)	Rational Flow Range (cfs)
353	898	3.5	.726	.507-.311	15.2	626-384
366	909	3.4	.730	.519-.321	16.0	646-399
599	1370	2.7	.847	.549-.356	22.5	887-576
655	1406	2.5	.859	.562-.372	25.2	920-609

Note: 1 in/hr = 25.4 mm/hr; 1 acre = 0.405 hectares;
1 cfs = 0.283 m^3/sec

Assuming the "C" values calculated using SWMM flows are correct, the peak rainfall intensities would have to be reduced to obtain the peak runoff rates generated using the Rational Method as follows:

Rational Method Flow Range (cfs)	Rainfall Intensity Range (in/hr)	Time of Concentration (minutes)
626 - 384	2.44 - 1.50	15.2
646 - 399	2.41 - 1.50	16.0
887 - 576	1.75 - 1.14	22.5
920 - 609	1.64 - 1.08	25.2

Extrapolating the above, using the appropriate intensity-duration-frequency curves indicates that use of the Rational formula results in protection from storms having a return frequency of about two years if maximum "C" values are used, to eight months if minimum "C" values are used. Hence, drainage systems designed by the Rational Method may not provide the anticipated degree of protection.

The magnitude by which the pipes are undersized can be estimated since the pipe diameter for a given rainfall intensity is directly related to the 3/8 power of the associated "C" if the slopes are maintained equal.

The following summary indicates the actual pipe diameter changes:

SWMM "C"	Rational "C" Range	SWMM Diameter (ft)	Rational Diameter Range (ft)
.726	.507-.311	8.0	7.0 - 5.8
.730	.519-.321	8.0	7.0 - 5.9
.847	.549-.356	11.5	9.8 - 8.2
.859	.562-.372	11.5	9.8 - 8.4

This demonstrates a pipe diameter underestimation of from 12.5 to 29 percent.

This undersizing of pipes can result in reduced capital costs but offers less protection against flooding. The following summary of typical costs for sewers installed in 18 to 20 foot trenches illustrates this apparent savings. The diameters are based on flows as calculated above and the costs are estimated using TABLE D-17.

SWMM Diameter (ft)	SWMM $/LF	Rational Diameter (ft)	Rational $/LF	Percent Savings
8.0	420.	7.0 - 6.0	350 - 298	17 - 29
8.0	420.	7.0 - 6.0	350 - 298	17 - 29
11.5	800.	10.0.- 8.5	612 - 452	3 - 44
11.5	800.	10.0 - 8.5	612 - 452	3 - 44

The inclination to use this method can be great since initial costs may be up to 44 percent lower. But use of the Rational Method to design storm sewers proposed to provide protection for the 5-year storm can result in flooding with storms having as short a return frequency as eight months.

As a further indication of the underestimation of flows generated by the Rational Method, reference is drawn to a drainage study in Louisville, Kentucky by the U.S. Army Corps of Engineers in 1949 (33). In this study report, a series of pumping station inflows were calculated, using both the Rational and the Unit Hydrograph approaches. The results of this comparison are summarized below:

Pumping Station	Rational Flow (cfs)	Unit Hydrograph Flow (cfs)
Seventeenth St.	99.9	126
Shawnee Park	939	1359
State Fair Grounds	1700	2520
Lower Paddy Run	1495	2630

The Rational flows are lower by 20 to 43 percent and the results are comparable to that predicted using the Elizabeth data.

METHODOLOGY

SWMM, as used in this work represents the state-of-the-art, hydraulically and hydrologically sophisticated, single storm event simulator. It uses a "micro" approach in that the rainfall-runoff process is divided into its basic elements. It simulates the process, spatially and temporally, and routes storm runoff and combined sewage quantity and quality from individual catchments and subcatchments through a sewer network, interceptor system, storage and treatment facilities, and finally to receiving waters. SWMM analyzes or designs a sewer system for an actual or synthetic storm event. It requires detailed delineation of the subcatchments and sewer layout and is an expensive procedure.

STORM is a continuous simulation model for overall planning. It analyzes hourly runoff quantity and quality for long-term hourly precipitation records, based on such parameters as percent imperviousness and land use and evaluates the effectiveness of storage and interceptor/treatment facilities for overflow pollution control. Typical to all continuous simulation models, STORM uses a "macro" approach by combining and averaging the effects of the basic elements. That is, rainfall-runoff processes are simulated for areas of finite size and for coarse time intervals. It emphasizes the gross pollution effect (total pounds) of pollutants discharged in a given storm rather than the detailed time and spatial characteristics. To operate on precipitation data over long time periods, the model contains a feedback mechanism which does continuous accounting of water inputs (rainfall and/or snowmelt) to and losses (depression storage and infiltration) from the drainage basin. In dealing with long-term real rainfall events, STORM was a necessary tool in the project study.

The methodology used follows:

1. SWMM developed the alternative designs in District A for runoff from a synthetic, intermediate pattern, 5-year return interval hyetograph shown in Figure D-5. Quantity and quality characteristics, with respect to combined sewage and storm runoff, were also determined for smaller rainfalls.

2. STORM evaluated the effectiveness of storage and interceptor/treatment facilities for storm runoff and combined sewage overflow pollution control using continuous real rainfall events.

3. Calibration of the quantity parameters contained in STORM used SWMM data generated from the detailed simulation of District A.

4. Quantity and quality and cost data, obtained from District A, were projected to the entire City for the evaluation of sewer system alternatives for both flood and pollution control.

APPENDIX B

MODIFICATION OF SWMM AND STORM PROGRAMS

In addition to continuously updating the models as revisions became available, both models were modified to meet the project requirements. The modifications made to SWMM and STORM and the necessity of making them are described.

SWMM

Both the RUNOFF and TRANSPORT Blocks in SWMM consider flow routing in sewer pipes. The RUNOFF Block employs a simple storage routing scheme using the continuity equation of flow in the gutters/pipes and Manning's equation. This technique is sufficient as the RUNOFF Block considers runoff in overland and gutter/pipes where sewer sizes are generally small and slopes are relatively steep. The RUNOFF Block can handle up to 200 pipe elements.

The sewer routing in the TRANSPORT Block uses a more complicated technique of a modified kinematic wave approach in accordance with Manning's and continuity equations. Backwater effects within individual sewer segments are considered. The TRANSPORT Block can handle up to a total of 160 sewer elements (conduit and non-conduit), of which about half are non-conduit elements. The TRANSPORT Block uses better routing techniques and has a sewer design option that the RUNOFF Block lacks. In the design version, small diameters are first assumed for the sewers that would ensure full pipe flow. The size of the sewer is then increased and the computation is repeated until free-surface flow occurs. In the previously available RUNOFF Block, pipes are permitted to surcharge when full. The surcharged volume is assumed to be temporarily stored at the upstream junction until sewer capacity becomes available to drain it. Use of the RUNOFF Block for design requires trial assumption of sewer sizes until stored surcharges are eliminated.

Because the SWMM RUNOFF Block (a) can handle more pipe elements than the TRANSPORT Block; and (b) has an acceptable flow routing routine, it appeared advantageous to use the RUNOFF Block for design of upstream lateral sewers. Consequently, SWMM was modified to allow design capability with pipes surcharged. As the capacity of the initially assumed pipe size is exceeded, the pipe size is automatically increased to the next larger commercial size until free-surface flow is attained. When a pipe is resized, the runoff computations are repeated for the new (larger) conduit for the current time step only. The runoff hydrograph up to the time when surcharging would have occurred in the particular pipe is not recomputed since this would require a basic change of the internal computational sequence. The design version of the TRANSPORT Block also does not recompute the previously computed hydrograph and pollutograph. The subroutines involved in the program changes include RHYDRO, and GUTTER.

STORM

The available version of STORM was developed for simulation of urban stormwater runoff quantity and quality. It did not model dry-weather flow and consequently could not be used for simulation of combined sewage overflows. Combined sewer overflow problems and their extent, and corrective measures, and their effectiveness, were an essential part of the project study. Hence, STORM was modified in this study to add the dry-weather flow simulation capability.

Dry-weather flow input data provides for diurnal variation of quantity and quality for various land uses. Dry-weather flow can include domestic, commercial and industrial wastes and infiltration. Four options are provided for computation of average daily dry-weather flow quantity:

1. inputs of the total dry weather flow,
2. inputs of flow for each land use separately,
3. inputs of the coefficients to be applied to populations or areas to estimate each component of the dry-weather flow. Domestic flow is assumed to be linearly proportional to population. Other component flows are linearly proportional to the area, and
4. use of default values provided in the program.

The daily flow may also be varied for weekdays, Saturdays, Sundays and Holidays and for hourly flow during the day by input of user-specified ratios or by default values in the program. This option allows computation of the diurnal flow variations.

For computation of dry-weather flow quality, coliform counts were added to the five pollutants considered in the available STORM program. Computation of coliform organisms washoff is assumed similar to the computation for other pollutants except that no contribution is assumed from the suspended and settleable solids. The default values for the coliform counts in dust and dirt are programmed. These default values, like those for other pollutants, are used to compute the amount of pollutant washout in relation to the amount of dust and dirt washout in a storm event. The options provided to compute average daily dry-weather flow pollutant loading for all six constituents are similar to that provided for the dry-weather flow quantity computation. Variations of pollutant load for the day of the week and hour of the day are also incorporated.

Changes to the STORM program include (a) the addition of Subroutine DWFLOW which carries out most of the dry-weather flow computation, (b) substantial modification of the MAIN program and Subroutine OUTPUT which reads input data and prints out computed results respectively, and (c) minor modifications to Subroutines DEFINE, DIRT, EVENT, and ZERO. A copy of the program changes was forwarded to the Hydrologic Engineering Center of the Army Corps of Engineers and was incorporated in the new version of STORM released in July, 1976 (28).

In the original version of STORM, a storm event is defined by the properties of the individual rainfall events, such as duration and intensity, storm spacing, and time to empty any storage facilities. An event was defined as beginning when storage was first required and continued until the storage basin was emptied. Any number of overflows occurring during the duration of the event was considered as the same overflow event. If a rainfall event produces runoff that does not exceed the sewer capacity, storage would not be utilized, and the rainfall would not register as an event. In the case of zero storage, runoff from watershed would directly drain to the interceptor sewer, and the number of events as defined above would be the same as the number of overflow events.

With the inclusion of dry-weather flows, the treatment or interceptor capacity should be at least adequate to handle peak dry-weather flows. Flows exceeding this value would require additional control measures.

APPENDIX C

QUANTITY AND QUALITY CONSIDERATIONS, SWMM VS. STORM

Difference between SWMM and STORM regarding storm runoff quantity and quality are worth noting. Recognition of these differences is necessary for interpretation and interchange of results generated by the two models.

QUANTITY

STORM accounts for losses of rainfall and/or snowmelt volume due to infiltration in the watershed by the use of a runoff coefficient. The runoff computation during a given hour is expressed as:

$$R = C\ (P - f) \qquad \text{(C-1)}$$

where R is runoff volume, P the rainfall volume, f the available depression storage and C the composite runoff coefficient depending upon urban land use. C is derived from two basic coefficients, C_1 and C_2. C_1 represents the runoff coefficient for pervious areas and C_2 for impervious areas. For a given watershed, knowing the area for each land use and the associated percent of imperviousness, C can be calculated arithmetically from C_1 and C_2. In the STORM Program, the default values of C_1 and C_2 are respectively 0.15 and 0.90. These coefficients are a function of rainfall characteristics (depth, duration, intensity) and antecedent dry conditions and calibration is required. The parameter "f" represents the antecedent conditions. There is no runoff from the watershed until depression storage, which is uniformly applied to the entire watershed, is filled.

In SWMM, runoff volume over a short time interval, measured in minutes, is determined by using a number of overland flow elements to simulate the initial collection processes. The amount of runoff from pervious and impervious areas is separately considered. Rainfall on certain impervious areas results in immediate runoff without loss to depression storage. This is the case in Elizabeth, since most dwellings have roof drains directly connected to a street gutter.

SWMM uses a more detailed concept of the hydraulics of rainfall runoff process than STORM. Parameters required can be reasonably estimated. Consequently, in the absence of real time runoff data in the study area, data generated by SWMM in District A were used to calibrate the runoff coefficients C_1 and C_2 in STORM.

QUALITY

Neglecting land surface erosion and dry-weather flow, both SWMM and STORM compute pollutant street washout in runoff according to the amount of dust and dirt (DD) accumulated along the street curbs prior to occurrence of a storm. From the total pounds of dust and dirt washout, the loadings of pollutant components such as SS and BOD are computed either from the available local data or from specified default values. Where quality data is not available for evaluating pollutant parameters, default values specified in either program could be used. TABLE C-1 shows the default values internally specified in SWMM and STORM for computation of SS, BOD and coliform loading from mass of dust and dirt in the street washout. Also shown in the table are the rates of dust and dirt accumulation in each land use. These rates were obtained from an APWA study in the Chicago urban area (34) and are used in both programs as default values.

TABLE C-1. POLLUTANT DEFAULT VALUES FOR SWMM AND STORM

Land Use	Dust and Dirt (DD)* lb/day/100 ft. of curb	mg Pollutant/ gram of DD SS SWMM	SS STORM	BOD*	Coliforms (MPN/gram)
Single Family	0.7	1000.	111.	5.0	1.3×10^6
Multiple Family	2.3	1000.	80.	3.6	2.7×10^6
Commercial	3.3	1000.	170.	7.7	1.7×10^6
Industrial	4.6	1000.	67.	3.0	1.0×10^6
Park	1.5	1000.	111.	5.0	0

* For both SWMM and STORM to compute soluble BOD

1 lb/day/100 ft = 1.486 kg/day/100m

The SWMM TRANSPORT Block only routes three pollutants; SS, BOD and total coliforms. These are commonly used as indicators of effluent water quality. In this study, only SS and BOD were used.

SWMM and STORM use the same default values for pollutant loading calculations except for SS and BOD. SWMM assumes the quantity of SS as about ten times greater than STORM. Both SWMM and STORM use the same values for computing pounds of soluble BOD from pounds of dust and dirt. However in the programmed calculations, SWMM adds five percent of the SS and STORM 10 percent to allow for the non-soluble BOD contribution. This results in BOD loadings (or concentrations) computed using SWMM default values equal to about five times that computed using STORM default values. The SS factors used by STORM are based on calibrations from the Selby Street data in San Francisco (35) rather than on physical analysis on dust and dirt. The SS factors used by SWMM appear to be based on convenient choice.

Test runs made by Huber (35) with Lancaster, Pennsylvania data, show that use of the SWMM SS default values and availability factor could produce SS concentration levels corresponding roughly to that obtained using the STORM default values. The availability factor varies with the rate of runoff and measures the portion of the dust and dirt available in a given rainfall event for the production of SS (6). Recently, Ganczarczyk (36) studied 99 hectares (245 acres) of catchment in Canada under developed conditions using the STORM program and STORM default values. The predicted BOD was of the same order of magnitude as reported in the literature for several cities both in and out of the United States while predicted SS were underestimated.

As SWMM is an event simulator and STORM a tool for runoff simulation of long-term precipitation records, the computations of street dust and dirt accumulation with varying numbers of antecedent dry days and street sweeping intervals are different. These differences are best illustrated by an example. Figure C-1, developed for District A, compares the predicted SS accumulation in streets using the SWMM and STORM programs with both SWMM and STORM default values for an assumed seven-day street sweeping interval and a 75 percent sweeping efficiency. The daily SS accumulation in streets of District A computed with the SWMM default values is 1,195 kg (2,638 pounds) and with the STORM default values, 131 kg (290 pounds). The SS accumulation computed in SWMM increases monotonically with the number of antecedent dry days. Those calculated with STORM indicate periodical fluctuation of SS accumulation at the street curb reflecting the effect of street cleaning frequency and the number of dry days since the last street cleaning. In SWMM, the additional accumulation of dust and dirt, consequently SS, on the street curb is assumed to be equal to the maximum accumulation for the period between successive street sweepings. It does not credit the cleaning effects of street sweeping. Recognition of this difference is significant when comparing runoff quality from a single storm event with two models.

In computing pounds of pollutants washed off streets in any given time interval, both SWMM and STORM assume the washoff rate to be proportional to pounds remaining on the ground and to the runoff rate. In SWMM, the rate is the average from both pervious and impervious areas. STORM uses runoff from impervious areas alone. For a given storm event, the runoff rate from an impervious area is greater than the average rate from both the impervious and pervious areas. A higher runoff rate means higher washoff and a greater availability factor. Consequently, for the same storm event, pollutant washout computed by STORM could be greater than that computed with SWMM.

Since STORM does not model sewers, it can not model solid deposition in sewers during dry days or their resuspension during wet days. For this reason, STORM can not simulate the "first flush" phenomenon in a combined sewer system.

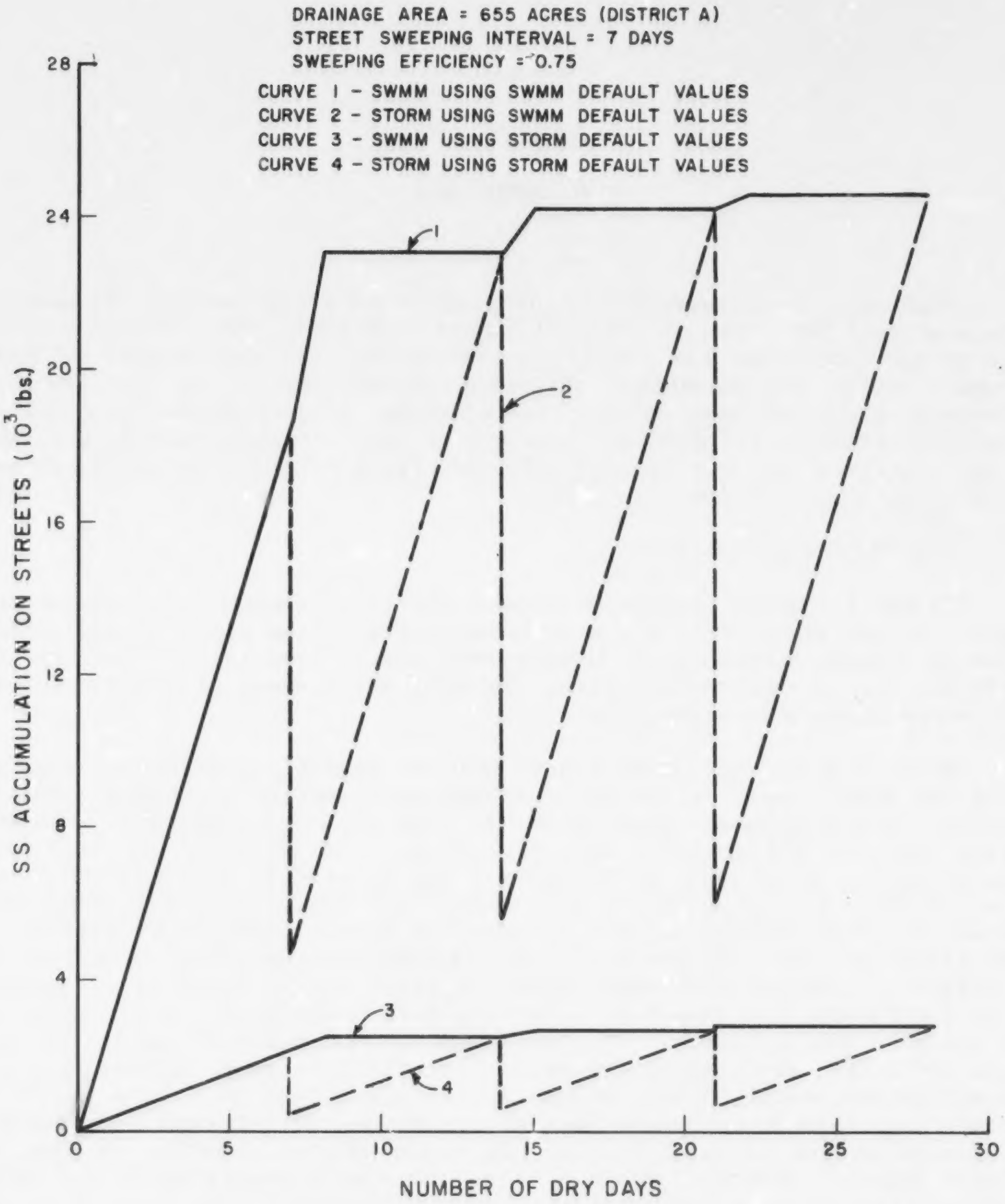


Figure C-1. Street SS accumulation, SWMM vs. STORM

APPENDIX D

MODEL INPUT DATA

Input data to computer models utilized in the study includes (a) meteorological data for long-term rainfall-runoff simulation, (b) a 5-year synthetic storm hyetograph for runoff determination, (c) hyetographs of less intense storms for planning a pollution control system, (d) the discrete elements of the drainage system, (e) hydraulic properties and land use of discrete elements, (f) area and land use of other drainage districts in the urban area, and (g) unit cost data for sewers of varying size and depth and other elements of the sewerage system.

LONG-TERM METEOROLOGICAL DATA

To use the STORM program to account for the rainfall-runoff characteristics of the study area, the continuous record of the hourly precipitation data at Newark International Airport over the 12-year period from 1963 to 1974 was used without modification. Rainfall was assumed to be uniform over the seven-square mile study area.

Average daily air temperature data at Newark International Airport over the same 12-year period was also used for snowfall and snowmelt computation. Precipitation occurring on the days with air temperatures at and below freezing (°C or 32°F) were considered to be snowfall and no runoff was assumed. Snow pack on the ground was assumed to start melting as the recorded air temperature rose above freezing. TABLE D-1 shows the amount of annual precipitation, the number of annual precipitation events and the estimated number of runoff events. A precipitation event is defined as a period of continuous non-zero precipitation. Traces (rainfall or equivalent water depth less than 0.25 mm/hour or 0.01 inches/hour are considered to be non-zero precipitation. Precipitations interrupted by a period of one hour or longer are separate events. To eliminate many inconsequential precipitation events, those events with a total less than 0.76 mm (0.03 inches) are deleted and become part of the dry period. The estimated number of runoff events is based on the depression storage capacity of 3.94 mm (0.155 inches) computed for District A. No runoff occurs until the total precipitation exceeds the depression storage capacity. The 12-year period selected for evaluation of stormwater runoff and combined sewage overflows included the severe drought years of the early 1960's and the very wet years of the early 1970's. The analysis made with the data collected during this period, therefore, should be representative of the range to be expected in future years.

TABLE D-1. ANNUAL PRECIPITATION AND RUNOFF EVENTS

Year	Precipitation (inches)	Number of Precipitation Events	Number of Runoff Events
1963	29.48	96	56
1964	33.45	118	76
1965	26.02	108	59
1966	37.86	94	51
1967	44.09	132	84
1968	36.89	89	53
1969	40.74	132	77
1970	34.38	113	74
1971	49.78	130	79
1972	48.30	135	83
1973	46.29	125	72
1974	39.86	111	70
Annual Average	38.89	115	70

Annual Average (1941-1970)=41.45 inches

Note: 1 inch = 25.4 mm

There are, on the average, about 115 annual precipitation events. About 40 percent of these precipitation events have a total less than the depression storage capacity for District A and would not result in runoff using STORM. About 70 annual storm runoff events are estimated for District A.

Figures D-1, D-2 and D-3 present a statistical analysis of the amount and duration of each precipitation event, and the antecedent dry hours for the 12-year period. Precipitation events can be expected on the average of about every three days with an average duration of about 4.5 hours. The median value of antecedent dry hours is 36, and precipitation duration, three hours. Similar analysis was made for cumulative precipitations exceeding 3.94 mm (0.155 inches) or greater than the assumed value of depression storage. The average interval between consecutive runoff events is about six days with a median of four days. The average duration of each runoff event is seven hours and the median duration is six hours. About 75 percent of the precipitation events with a cumulative rainfall of less than 3.94 mm (0.155 inches) were of three hours or less duration, and about 90 percent were of four hours or less. During the 12-year period, the longest duration rainfall was 34 hours (September, 1969), with a cumulative total of 126 mm (4.97 inches). The largest amount of precipitation was 143 mm (5.62 inches) over 15 hours recorded on August 29, 1971. In October, 1973, 26 consecutive dry days were recorded, the longest in the 12-year period.

Previous study (8) of rainfalls in the Elizabeth River drainage basin indicated about 35 percent of the rainfalls were of the advanced pattern

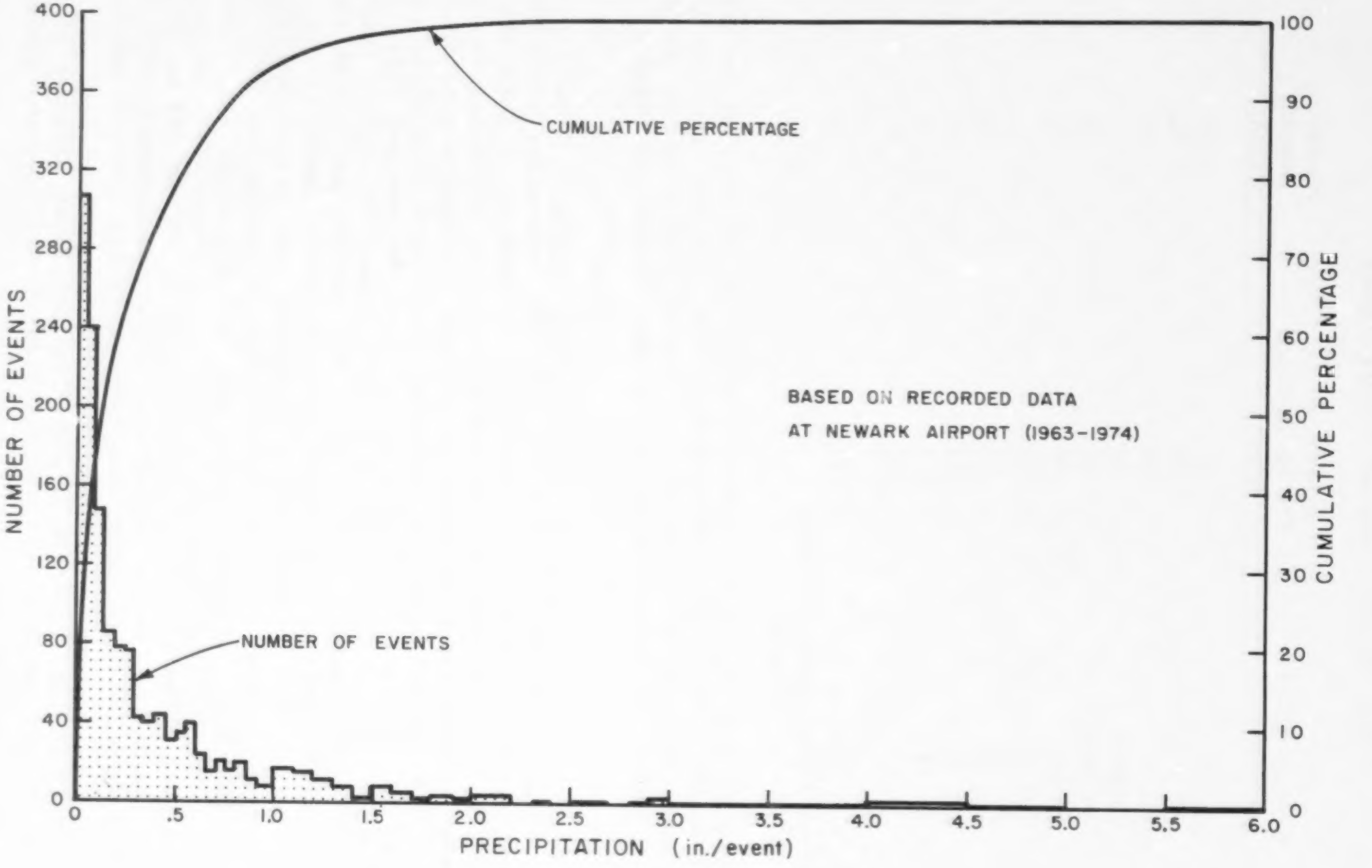


Figure D-1. Frequency distribution of precipitation amounts

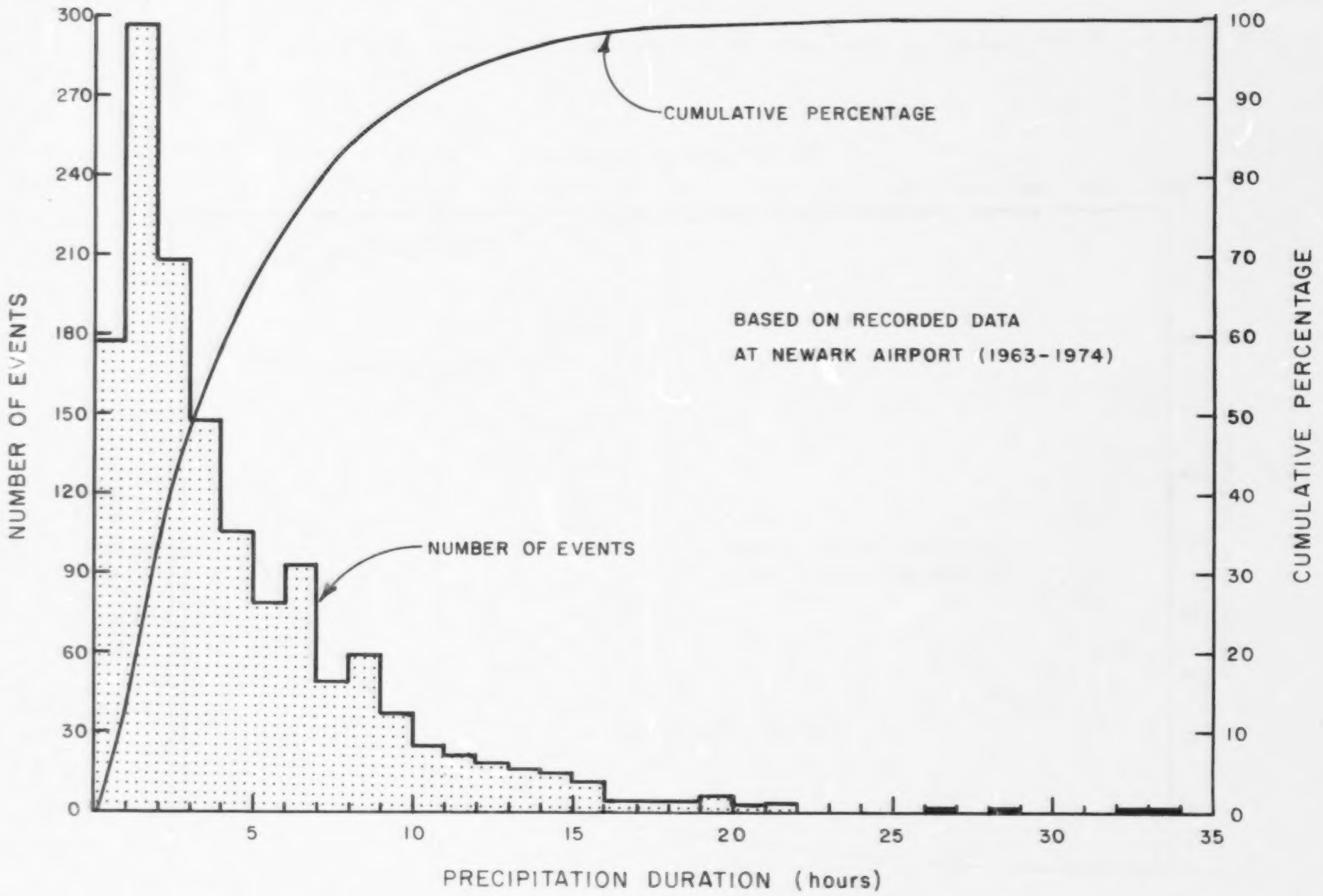


Figure D-2. Frequency distribution of precipitation duration

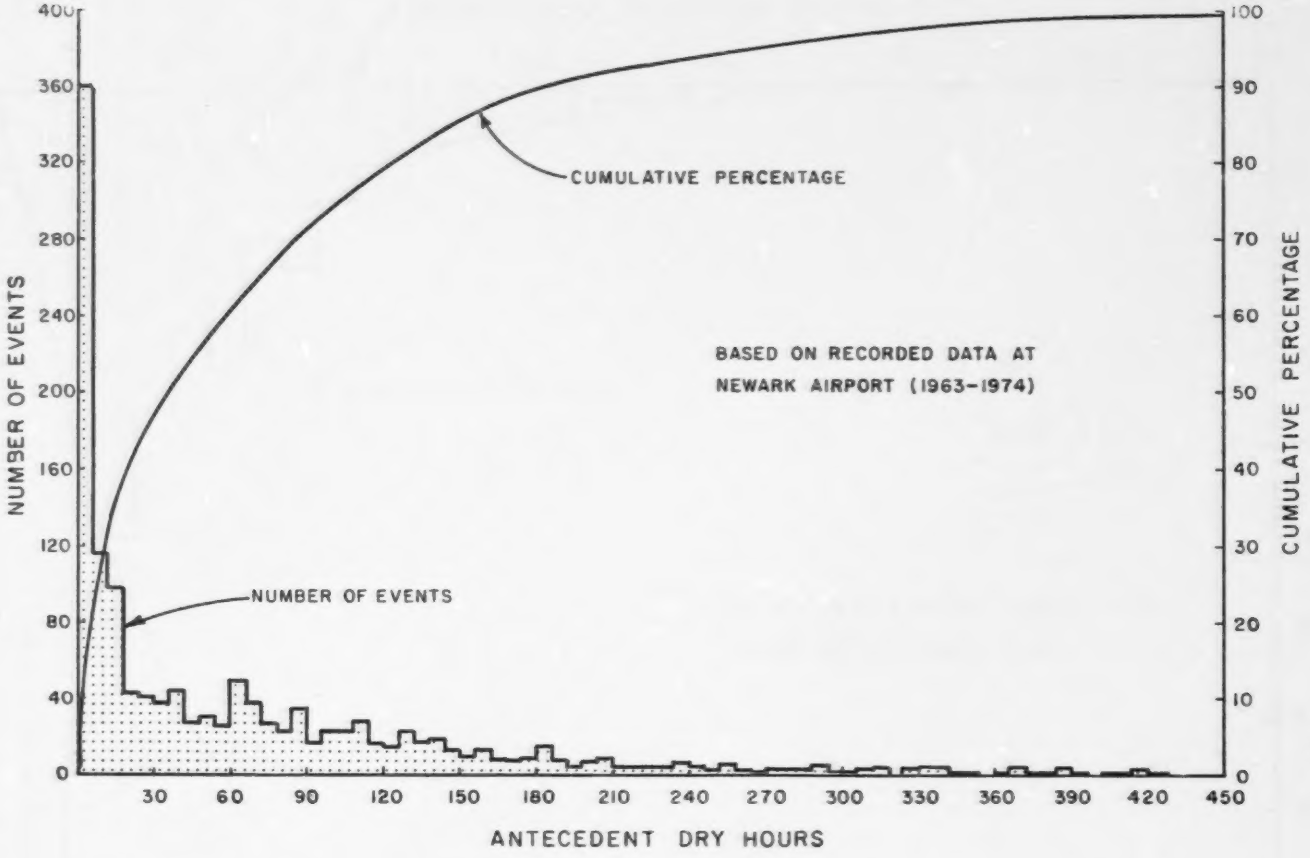


Figure D-3. Frequency distribution of antecedent dry hours in which the most intense rainfalls occurred in the first quarter of the storm period, about 45 percent were of the intermediate pattern in which the peak rainfalls occurred in the middle half of the storm, and 20 percent were of the delayed pattern in which peak rainfalls occurred in the last quarter of the storm period. The 12-year data generally conforms to this rainfall pattern. Peak runoff from an advanced pattern storm is less than that from an intermediate or delayed pattern storm since the rainfall-producing runoff, or rainfall excess at the time of the peak rate of precipitation is less due to depression storage and greater infiltration losses. In this study, both the advanced and intermediate pattern storms are considered in evaluating control of wet-weather overflows.

FIVE-YEAR SYNTHETIC STORM HYETOGRAPH

The sewer system was designed to accept runoff from an intermediate pattern synthetic storm hyetograph. The hyetograph of the synthetic storm was developed using the intensity-duration curves shown in the U.S. Weather Bureau Technical Paper No. 25 (37) for Sandy Hook, New Jersey. The intensity-duration curves shown in Figure D-4 were considered to be applicable to the study area. Twelve-year data at the Newark International Airport was analyzed. The peak hourly rainfall with 5-year return interval was in excellent agreement with that obtained from the Sandy Hook intensity-duration curves. The rainfall intensity-duration curves at Sandy Hook were updated in a more recent U.S. Weather Bureau publication, Technical Paper No. 40 (38). Intensities obtained from this later publication were found to be about 10 percent greater than that obtained from the older Technical Paper No. 25.

The synthetic storm hyetograph was computed for the assumed duration of three hours, with peak rainfall intensity assumed to occur at the beginning of the middle third of the storm duration. This duration is long enough to cover the time of travel from headwater to outlet in the largest district in Elizabeth.

In developing the shape of the storm hyetograph, it was recognized that real storm events can produce rainfall intensities equal to different return frequencies during their history. In the storm of July 20, 1961 at the Newark International Airport Weather Station, the frequency approximated a 15-year return period storm at ten minutes duration, a two-year return period at one hour duration and less than this return period for three hours duration. The measured data is shown in Figure D-4. Because each reach of a sewer conveying urban drainage is tested by a different duration storm for a given return frequency, the concept of an envelope hyetograph has been developed and successfully applied. The intensity-duration relationships of this hyetograph matches the Weather Bureau intensity-duration curves at all time intervals. This hyetograph can be used to determine the design flows which test the entire system, and eliminates the need of using more than one hyetograph of varying durations in design.

Figure D-5 shows the 5-year synthetic storm hyetograph developed for 2-minute intervals. The storm hyetograph was developed as follows. The intensities were read from the intensity-duration curve for 5-year

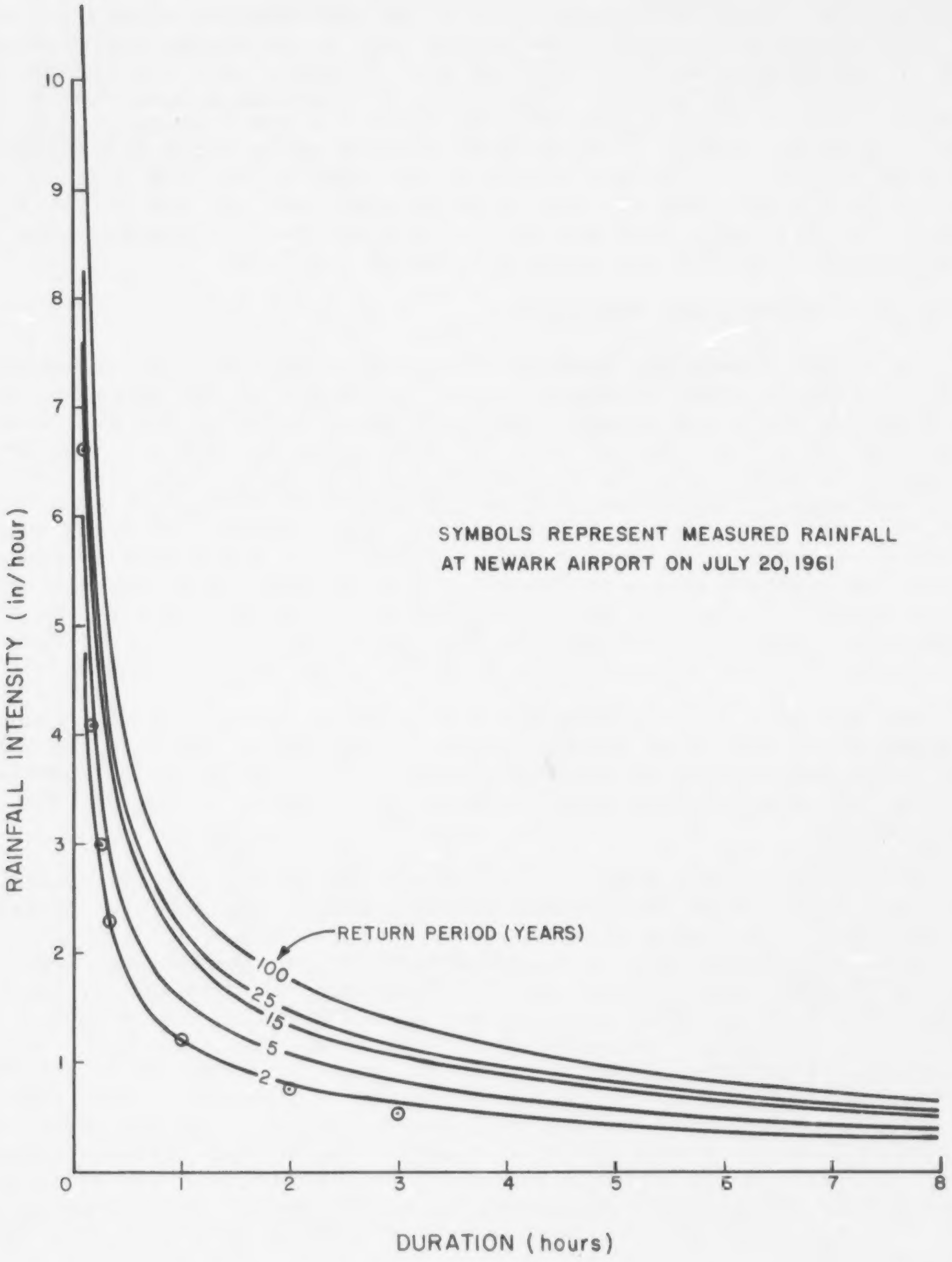


Figure D-4. Rainfall intensity-duration frequency curves

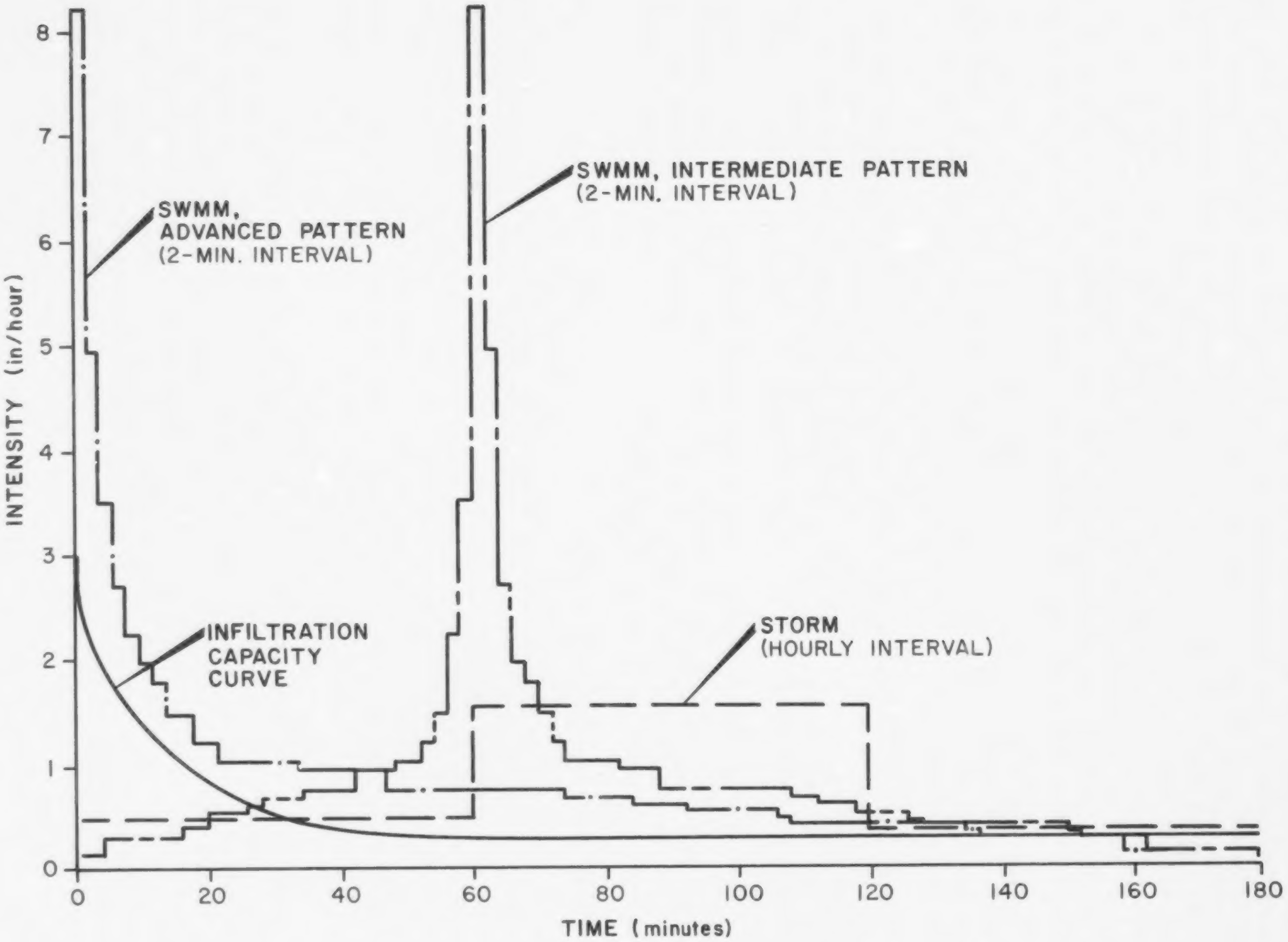


Figure D-5. 5-year design storm hyetographs and infiltration curve return frequency at durations which are multiples of two minutes. The rainfall amounts for incremental duration were obtained by multiplying the intensity and duration. The rainfall amount differential of a successive duration represents a possible 2-minute interval rainfall in the hyetograph. After all possible two-minute rainfalls have been obtained, they are recorded in sequence beginning with the largest two-minute interval rainfall. The most intense rainfall rate was placed at the beginning of the second hour of the hyetograph, the second most intense rainfall at the next later time interval and the third most intense rainfall at the next earlier time interval. The remaining hyetograph was developed in a similar fashion.

From the intensity-duration relation used to develop the 5-year synthetic storm hyetograph, the average 1-hour, 2-hour, and 3-hour rainfalls have intensities of 40.6, 26.7 and 20.6 mm (1.6, 1.05 and 0.81 inches) per hour respectively.

For input to calibrate the STORM program, which accepts hourly rainfall data only, the 5-year synthetic storm hyetograph with hourly intervals (Figure D-5) was used. The hourly intensities of the hyetograph are 12.7, 40.6 and 8.4 mm (0.50, 1.60 and 0.33 inches) per hour respectively. These intensities were computed so as to conform to intensity-duration curve.

Storms of advanced pattern were considered in relation with the containment or interception of storm runoff for pollution control. The advanced pattern storm was assumed to start with the most intense rainfall and then to decrease monotonically as the storm progressed. The 5-year synthetic storm of advanced pattern is shown in Figure D-5.

HYETOGRAPHS OF LESS INTENSE STORMS

The 5-year synthetic storm hyetograph was used for the design of collection systems for flood relief. Optimum sizing of the pollution control system components, such as interceptors, storage and treatment facilities, are defined here by less intense but more frequent rainfalls. Consequently, 1-year and 1.3-month synthetic storms were developed and used as input to SWMM to determine the character of runoff quantity and quality required to develop pollution control strategy. The hourly rainfall intensities of these 1-year and 1.3-month synthetic storms and storms of other return intervals are shown in TABLE D-2.

The data were prepared from a partial duration series analysis of hourly data for the 12-year period assuming hourly intense rainfalls were independent of others regardless of whether they occurred in the same precipitation event or in a separate event. Monthly exceedance values of the rainfall data were arranged according to magnitude. The values are plotted using the Weilbull formula (39) on semilog paper with the rectangular scale representing hourly rainfall intensities and the logarithmic scale representing the return interval in months. Figure D-6 shows such a plot. The plotted values lie approximately on a straight line up to the return interval

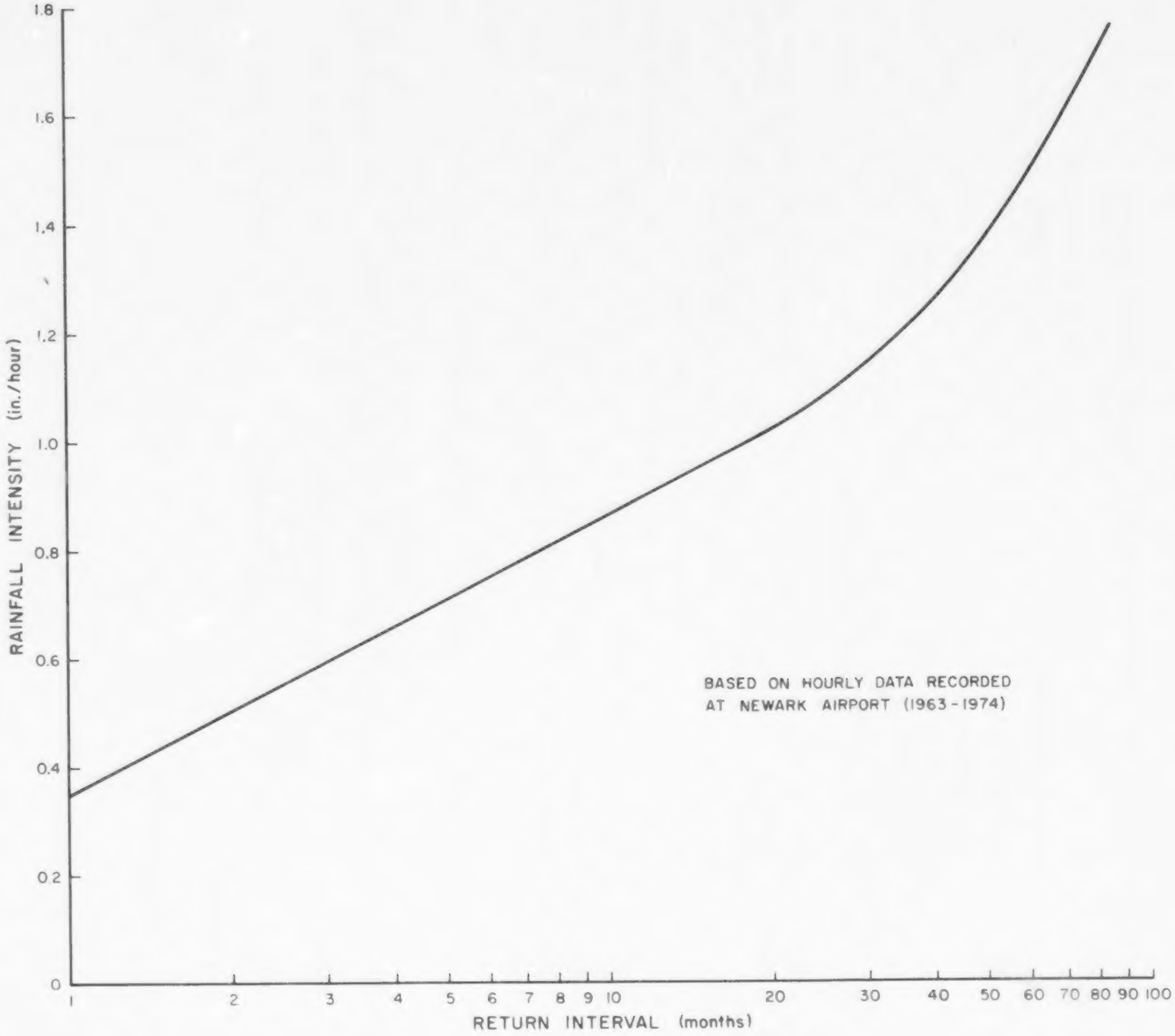


Figure D-6. Frequency of hourly rainfalls of two years. To check the hourly rainfall of 40.6 mm (1.6 inches) of the 5-year return interval storm, an annual exceedance series was prepared and plotted on a Gumbel's extreme probability paper. The hourly rainfall value corresponding to the 5-year return interval agreed well with 40.6 mm (1.6 inches).

TABLE D-2. HOURLY RAINFALL VERSUS RETURN INTERVAL

Return Interval	Hourly Rainfall (inches)
6 - Month	0.74
3 - Month	0.60
1.3 - Month	0.40
1 - Month	0.34

Note: 1 inch = 25.4 mm

The advanced and intermediate rainfall hyetographs of 1-year and 1.3-month storms were synthesized in the same manner as the 5-year synthetic storm. The magnitude of these storm hyetographs were derived from the 5-year synthetic storm hyetograph by multiplying the latter by the ratios of 0.58 (0.92/ 1.60) and 0.25 (0.40/1.60) (TABLE D-2) respectively for the 1-year and 1.3-month synthetic storms. TABLE D-3 summarizes the gross characteristics of 5-year, 1-year and 1.3-month synthetic storms. The 1-year and 1.3-month synthetic storms as developed represent one of many possible storm patterns. However, they provide a consistent tool for the derivation of runoff quantity and quality information for planning level decisions of pollution control strategy.

TABLE D-3. SYNTHETIC STORM CHARACTERISTICS

Return Interval	Average Rainfall Intensity (in/hr) 1-hour	2 hour	3-hour	Total Rainfall (inches)
5-Year	1.6	1.08	0.81	2.43
1-Year	0.92	0.63	0.47	1.41
1.3-Month	0.40	0.27	0.20	0.60

Note: 1 in/hr = 25.4 mm/hr

DEFINITION OF DISCRETE ELEMENTS

For use of SWMM, District A was divided into five major subareas, each having relatively homogeneous land use. These five subareas are shown in Figure D-7. Subareas I and II are almost entirely single family residential housing, which contain approximately 43 percent of impervious surface. About half of Subarea III consists of commercial and industrial development which contains about 80 percent impervious area. Subarea IV includes a park with about 19 percent impervious area, while Subarea V is generally high-density residential with 50 percent impervious area. TABLE D-4 summarizes the land use distribution for each subarea as well as for all of District A.

TABLE D-4. AREA OF LAND USE DISTRIBUTION IN EACH SUBAREA (ACRES)

Subarea	Single Family	Multiple Family	Commercial	Industrial	Parks	Total Area	Percent Imperviousness
I	175.73	-	-	-	-	175.73	43.0
II	164.59	11.60	0.33	-	-	176.52	43.5
III	9.18	43.03	27.07	18.02	-	97.29	63.2
IV	115.36	17.76	4.67		13.03	150.83	42.9
V	5.33	46.83	2.63	-	-	54.79	50.8
Total	470.19	119.22	34.70	18.02	13.03	655.16	46.8

Note: 1 acre = 0.405 hectares

The five subareas were further subdivided into 279 subcatchments with areas ranging from 0.08 to 4.05 hectares (0.2 to 10 acres) with an average of 0.93 hectares (2.3 acres). Overland drainage was routed to the nearest street. The surface runoff drained to 139 lateral sewers (analyzed with the SWMM RUNOFF Block) and subsequently to 31 trunk sewers (analyzed with the SWMM TRANSPORT Block). The downstream end of the sewer system in District A connects to the Westerly Interceptor. Flows exceeding the capacity of the interceptor discharge to the Elizabeth River.

The schematics of the combined sewer layout for each subarea are shown separately in Figures D-8 to D-12. The combined sewer system was extended from the separate storm sewer system shown in Figure D-13. It includes a pipe in every street but excludes drainage at the upstream end of each sewer branch. The separation of sewers in District A was studied previously (11) in an effort to alleviate frequent street and cellar flooding. In the separate system, overland flow was allowed to the maximum extent practical. TABLE D-5 compares the total length of separate sanitary, separate storm and the combined systems.

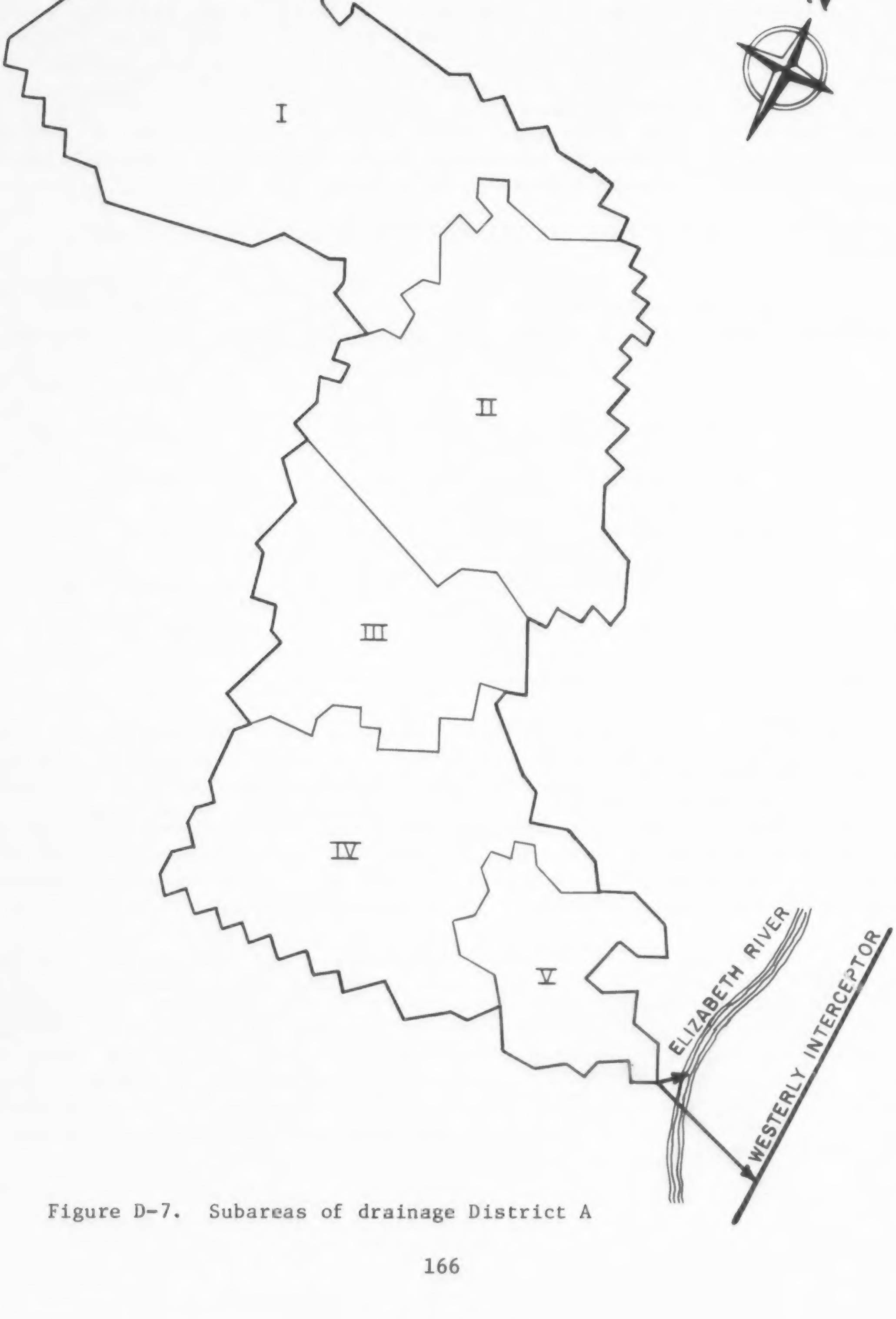
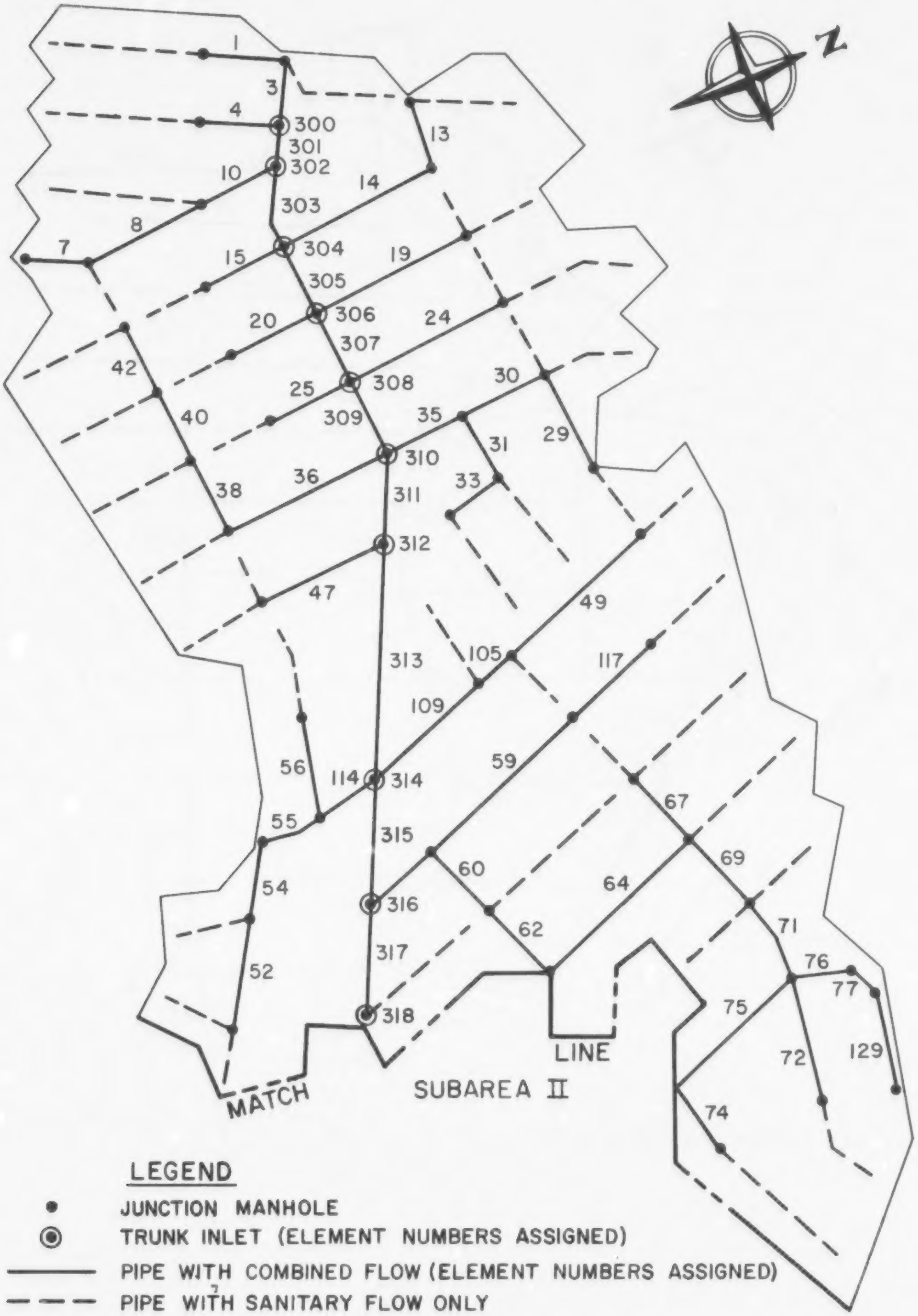

N
I
II
III
IV
V
ELIZABETH RIVER
WESTERLY INTERCEPTOR

Figure D-7. Subareas of drainage District A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301
302
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14
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31
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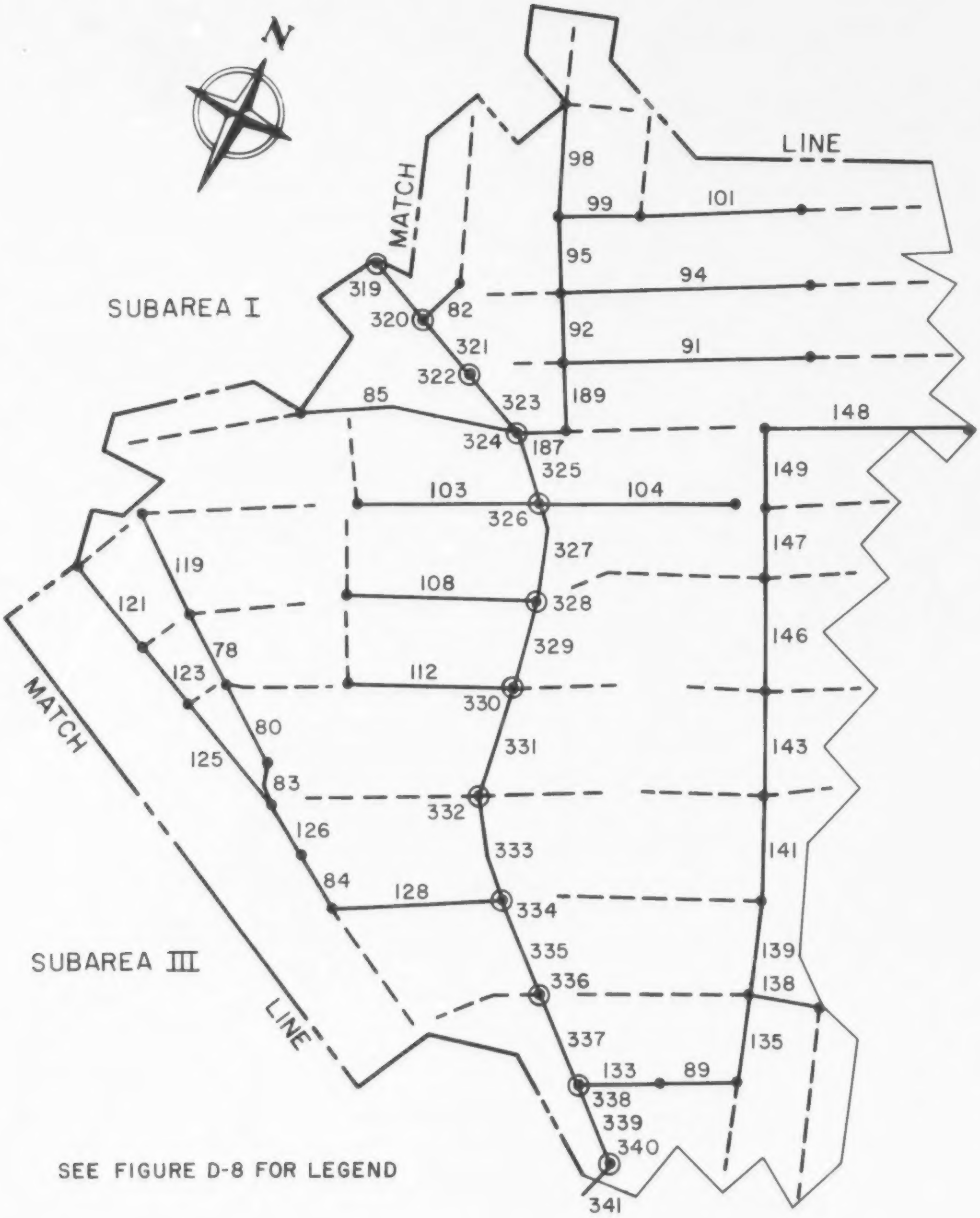
36
38
33
311
312
47
49
313
105
117
109
59
56
114
314
67
55
315
69
60
54
64
316
62
71
317
76
77
52
75
318
72
129
LINE
MATCH
SUBAREA II
74
LEGEND
● JUNCTION MANHOLE
◉ TRUNK INLET (ELEMENT NUMBERS ASSIGNED)
PIPE WITH COMBINED FLOW (ELEMENT NUMBERS ASSIGNED)
PIPE WITH SANITARY FLOW ONLY

Figure D-8. Combined sewer system layout, subarea I


N
MATCH
LINE
SUBAREA I
SUBAREA III
MATCH
LINE
98
99
101
95
94
92
91
189
148
319
320
82
321
322
85
323
324
187
325
103
104
326
149
119
327
147
121
108
328
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78
329
123
112
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80
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143
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126
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141
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128
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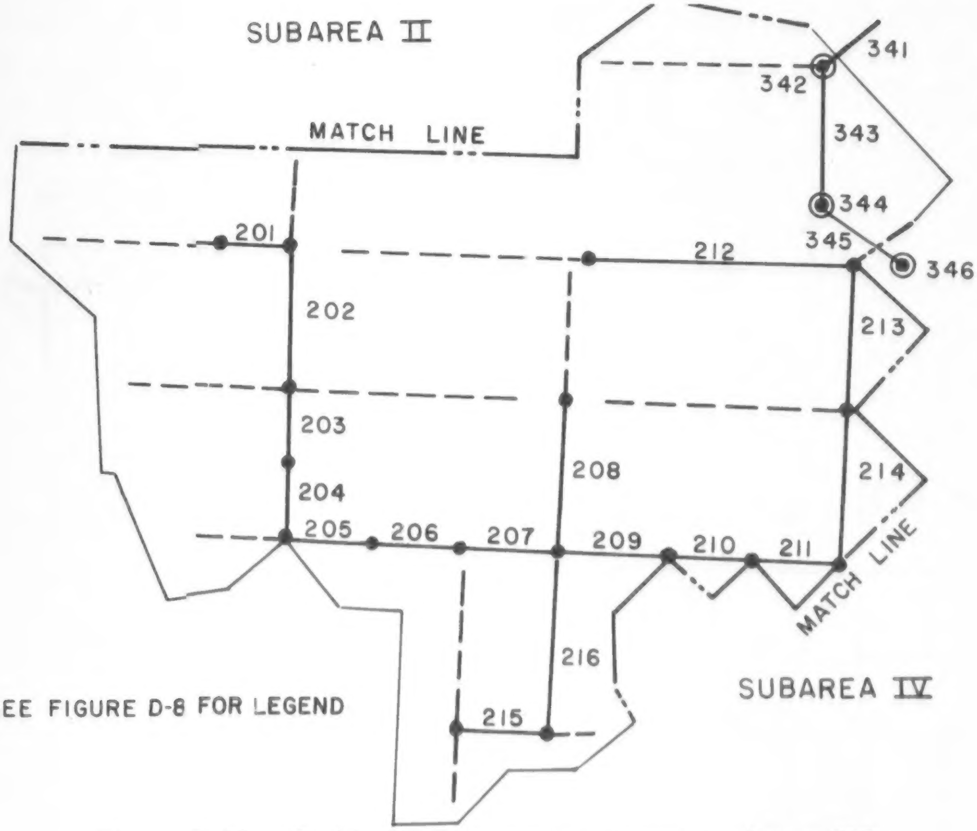
341

SEE FIGURE D-8 FOR LEGEND

Figure D-9. Combined sewer system layout, subarea II


SUBAREA II
MATCH LINE
341
342
343
344
345
346
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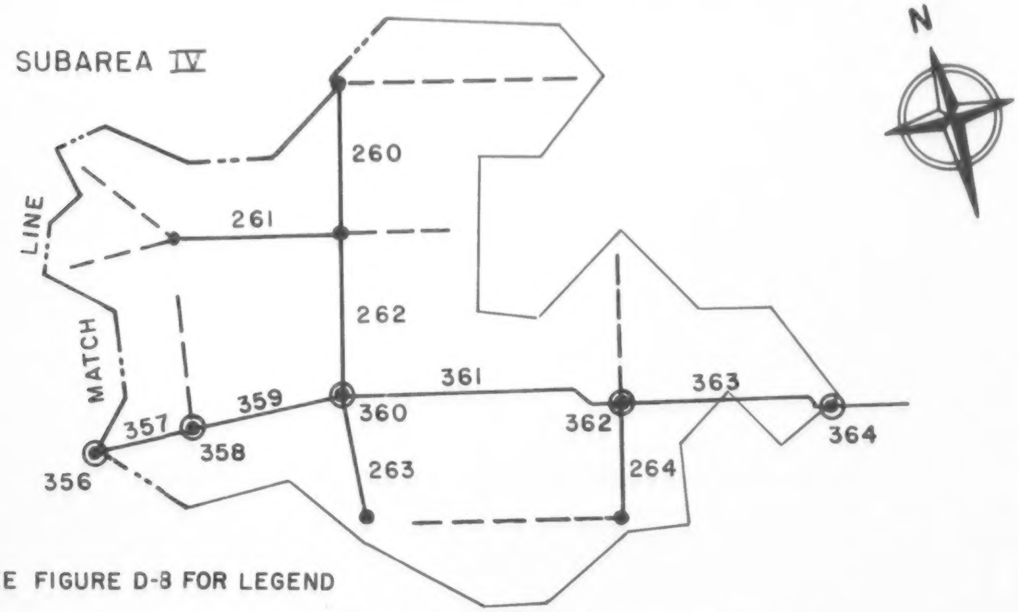
202
203
204
205
206
207
208
209
210
211
212
213
214
215
216
MATCH LINE
SUBAREA IV

SEE FIGURE D-8 FOR LEGEND

Figure D-10. Combined sewer system layout, subarea III


SUBAREA IV
N
260
261
262
263
264
MATCH LINE
356
357
358
359
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362
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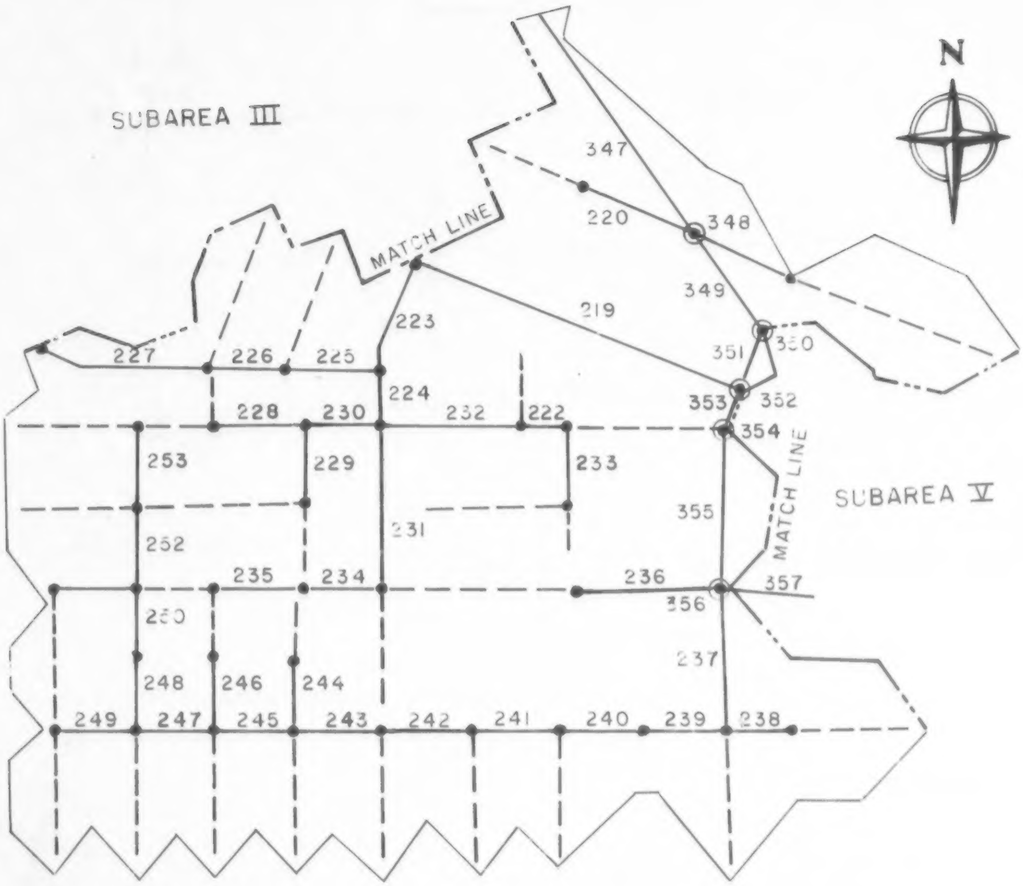
364

SEE FIGURE D-8 FOR LEGEND

Figure D-11. Combined sewer system layout, subarea V


N
SUBAREA III
MATCH LINE
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MATCH LINE
SUBAREA V
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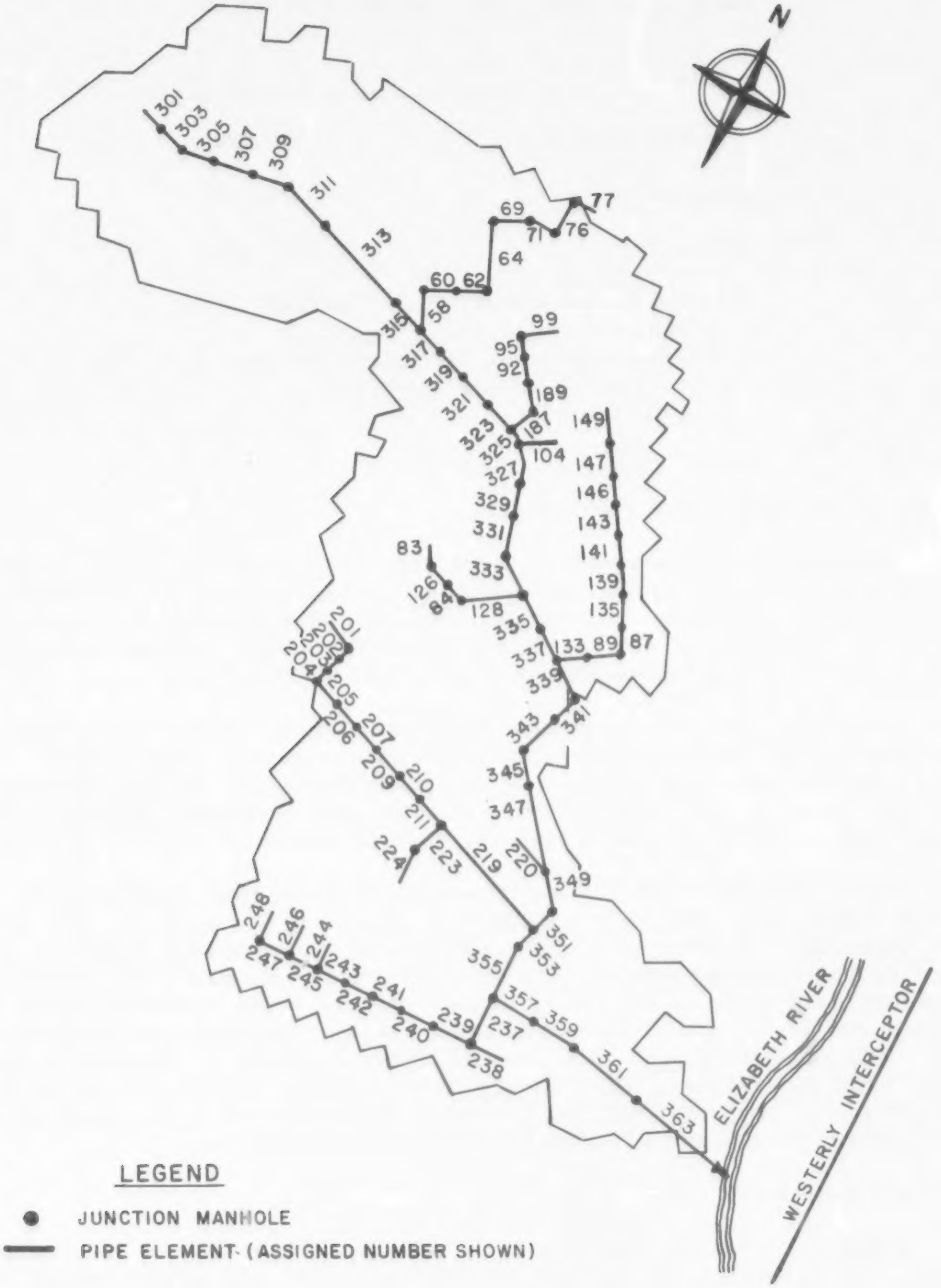
Figure D-12. Combined sewer system layout, subarea IV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ELIZABETH RIVER
WESTERLY INTERCEPTOR
LEGEND
● JUNCTION MANHOLE
PIPE ELEMENT (ASSIGNED NUMBER SHOWN)

Figure D-13. Separate storm sewer system layout

TABLE D-5. TOTAL SEWER LENGTH COMPARISON

System	Length (ft)
Separate Sanitary	106,000
Separate Storm	29,000
Combined	63,640*

* Length of sewers carrying combined sewage only. The total length of the combined sewer system, including extensions at extremities, is 106,000 feet.

Note: 1 foot = 0.3048 meters

HYDRAULIC PROPERTIES AND LAND USE

Delineation of subcatchment boundaries were prepared from a street map of scale 1:4,800 and field checked. Subcatchment areas were planimetered. Data prepared for each subcatchment included catchment width, area, imperviousness, slope, overland resistance factors, surface storage and infiltration factors. The subcatchment data are shown in TABLE D-6.

Definition of pipe elements includes length, slope, tributary area, upstream and downstream connecting pipes and roughness coefficients. TABLE D-7 lists the pipe element and its corresponding slope, length and tributary drainage area. The trunk sewer elements were assigned numbers starting at 300 and the lateral sewer elements, numbers below 300.

Since the SWMM RUNOFF (modified) and TRANSPORT Blocks have sewer design capability, the sewer diameters required as input to the program were generally assigned as 0.457 m (18 inches), the minimum diameter used in practice for combined sewers. The Manning "n" was assumed equal to 0.013 for diameters less than 0.762 m (30 inches) and 0.011 for diameters equal to or greater than 0.762 m (30 inches), respectively. The modified version of the SWMM RUNOFF program has incorporated these changes of roughness coefficient with pipe sizes. Data on the land use and curb length in each subcatchment was prepared and is presented in TABLE D-8. Dry-weather flows were entered at designated inlet manholes to trunk sewers.

DISTRICT A DATA FOR STORM

STORM is a "macro" model and is intended to consider a large drainage basin. It processes the composited information of all subcatchments rather

TABLE D-6. SUBCATCHMENT DATA FOR SWMM

SUBCATCHMENT NO.	GUTTER OR INLET	WIDTH (FT)	AREA (AC)	PERCENT IMPERV.	SLOPE (FT/FT)	RESISTANCE FACTOR IMPERV.	RESISTANCE FACTOR PERV.	SURFACE STORAGE (IN) IMPERV.	SURFACE STORAGE (IN) PERV.	INFILTRATION RATE (IN/HR) MAXIMUM	INFILTRATION RATE (IN/HR) MINIMUM	DECAY RATE (1/SEC)
1	126	262.0	5.06	48.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
2	64	390.0	2.24	47.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
3	84	323.0	2.2[illegible]	46.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
4	125	613.0	3.52	49.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
5	123	378.0	2.17	48.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
6	121	629.0	3.56	48.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
7	121	270.0	0.62	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
8	119	624.0	1.72	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
9	78	264.0	0.37	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
10	78	431.0	0.95	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
11	80	157.0	0.16	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
12	80	355.0	2.36	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
13	332	1084.0	4.46	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
14	128	845.0	3.49	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
15	336	639.0	2.20	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
16	338	436.0	1.50	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
17	336	566.0	1.95	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
18	336	503.0	1.50	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
19	334	368.0	1.66	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
20	332	795.0	3.10	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
21	332	401.0	1.64	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
22	112	731.0	3.19	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
23	112	650.0	2.24	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
24	112	363.0	1.25	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
25	119	711.0	2.94	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
26	119	958.0	3.30	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
27	108	454.0	1.2[illegible]	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
28	108	1013.0	3.49	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
29	330	327.0	1.50	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
30	330	632.0	2.90	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
31	328	372.0	1.28	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
32	103	898.0	3.36	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
33	103	272.0	1.25	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
34	85	1022.0	3.52	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
37	85	1467.0	4.04	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
38	104	1318.0	3.63	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
39	187	1398.0	3.8[illegible]	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
40	187	278.0	0.51	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
41	189	334.0	0.32	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
42	189	605.0	1.39	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
43	322	1002.0	1.64	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
44	320	479.0	1.76	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
54	82	1245.0	4.00	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
55	82	222.0	0.51	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
56	92	553.0	1.6[illegible]	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
57	92	366.0	0.84	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
58	91	2465.0	6.75	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
59	94	2705.0	6.63	43.0	0.0400	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
60	101	1659.0	4.57	43.0	0.0[illegible]00	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
61	99	584.0	1.61	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
62	95	286.0	0.92	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
63	98	573.0	1.84	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
64	98	366.0	0.84	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
65	99	479.0	1.65	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
74	96	529.0	1.38	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
165	340	659.0	2.24	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
164	89	374.0	1.63	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
163	133	404.0	1.39	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
162	69	310.0	1.28	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
161	89	254.0	0.70	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
160	138	263.0	0.73	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
159	87	265.0	0.73	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
158	135	534.0	1.64	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
157	135	240.0	0.55	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
156	138	316.0	0.73	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
155	139	307.0	5.10	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
154	139	295.0	0.88	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
153	141	795.0	3.10	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
152	141	432.0	1.65	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
151	143	632.0	2.90	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138

TABLE D-6. (continued)

SUBCATCHMENT NO.	GUTTER OF INLET	WIDTH (FT)	AREA (AC)	PERCENT IMPERV.	SLOPE (FT/FT)	RESISTANCE IMPERV.	FACTOR PERV.	SURFACE STORAGE(IN) IMPERV.	PERV.	INFILTRATION RATE(IN/HR) MAXIMUM	MINIMUM	DECAY RATE (1/SEC)
115	306	294.0	0.81	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
116	24	972.0	2.90	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
117	24	401.0	0.52	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
118	19	425.0	1.17	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
119	19	247.0	2.61	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
120	306	359.0	0.66	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
121	20	937.0	3.01	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
122	42	318.0	0.73	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
123	40	868.0	2.39	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
124	42	700.0	2.09	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
125	8	472.0	1.10	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
126	15	1107.0	3.05	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
127	14	985.0	3.38	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
128	19	359.0	0.99	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
129	19	475.0	1.65	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
130	13	567.0	2.02	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
131	13	351.0	1.21	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
132	3	640.0	2.94	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
133	302	543.0	1.87	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
134	10	265.0	0.73	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
135	8	708.0	1.95	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
136	7	553.0	1.65	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
137	10	1167.0	3.05	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
138	4	1450.0	4.66	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
139	3	278.0	0.51	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
140	1	1347.0	3.71	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
172	3	163.0	0.15	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
166	129	410.0	1.60	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
170	74	670.0	2.70	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
310	342	1427.0	6.20	67.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
309	344	455.0	2.10	80.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
306	344	440.0	2.00	80.0	0.0100	0.013	0.250	0.026	0.250	3.00	0.28	0.00138
283	215	1102.0	5.10	44.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
284	216	741.0	2.71	45.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
282	216	682.0	2.40	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
286	210	401.0	1.84	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
285	209	455.0	2.09	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
294	207	369.0	1.61	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
295	206	479.0	2.20	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
296	205	559.0	2.31	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
300	205	692.0	2.86	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
298	204	348.0	1.28	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
299	203	374.0	1.39	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
301	203	896.0	5.14	40.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
297	203	1531.0	6.68	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
293	208	560.0	2.57	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
274	214	486.0	2.68	74.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
287	214	1309.0	6.31	56.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
288	213	578.0	2.39	80.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
302	202	677.0	2.02	55.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
303	212	1420.0	8.15	80.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
304	201	1488.0	8.86	80.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
305	202	431.0	1.98	80.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
291	212	1458.0	8.37	80.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
292	208	584.0	2.68	65.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
201	249	694.0	2.39	45.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
202	247	746.0	2.57	45.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
203	245	739.0	2.28	45.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
204	243	765.0	2.46	45.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
205	242	669.0	2.61	52.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
206	241	649.0	2.66	54.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
207	240	741.0	2.72	54.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
208	237	623.0	2.86	59.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
209	238	1106.0	5.84	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
218	237	509.0	2.57	31.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
219	239	367.0	2.02	33.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
220	240	454.0	2.50	30.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
221	241	399.0	2.20	28.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
222	242	367.0	2.02	31.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
223	231	598.0	2.64	31.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138

TABLE D-6. (continued)

SUBCATCH-MENT NO.	GUTTER OR INLET	WIDTH (FT)	AREA (AC)	PERCENT IMPERV.	SLOPE (FT/FT)	RESISTANCE IMPERV.	FACTOR PERV.	SURFACE STORAGE(IN) IMPERV.	SURFACE STORAGE(IN) PERV.	INFILTRATION RATE(IN/HR) MAXIMUM	INFILTRATION RATE(IN/HR) MINIMUM	DECAY RATE (1/SEC)
150	143	351.0	1.61	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
149	141	383.0	2.32	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
148	143	534.0	1.84	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
147	146	1004.0	4.15	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
146	146	358.0	1.65	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
145	146	617.0	1.84	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
144	147	267.0	0.32	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
143	147	640.0	1.91	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
142	149	319.0	0.68	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
141	148	1295.0	3.27	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
171	101	552.0	2.12	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
35	52	958.0	2.20	43.0	0.0100	0.013	0.025	0.062	0.250	3.00	0.28	0.00138
36	52	521.0	2.33	43.0	0.0300	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
45	318	817.0	3.19	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
46	52	303.0	1.33	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
47	52	201.0	0.37	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
48	54	575.0	1.32	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
49	54	160.0	0.55	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
50	55	440.0	2.32	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
51	316	736.0	1.64	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
52	58	372.0	1.26	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
53	318	1027.0	3.30	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
66	74	583.0	2.28	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
67	62	419.0	1.25	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
68	62	345.0	1.03	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
69	60	332.0	0.99	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
70	109	834.0	2.66	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
71	59	1205.0	4.15	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
72	60	1002.0	3.45	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
73	64	501.0	1.84	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
75	75	755.0	3.12	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
76	72	1198.0	4.95	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
77	76	395.0	1.36	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
78	71	497.0	1.14	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
79	69	587.0	2.02	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
80	64	629.0	2.02	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
81	59	392.0	1.17	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
82	67	399.0	1.10	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
83	67	425.0	1.17	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
84	109	507.0	1.28	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
85	109	523.0	0.84	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
86	49	629.0	2.31	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
87	117	1194.0	4.11	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
88	67	340.0	1.20	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
89	54	898.0	3.30	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
90	69	671.0	2.31	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
91	49	363.0	1.25	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
92	56	1130.0	4.15	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
93	314	1182.0	4.07	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
94	109	832.0	1.91	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
95	33	861.0	2.37	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
96	31	933.0	2.57	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
97	29	799.0	2.75	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
98	31	319.0	0.86	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
99	33	338.0	0.99	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
100	312	399.0	1.10	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
101	47	580.0	2.13	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
102	47	281.0	0.84	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
103	47	501.0	1.61	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
104	36	574.0	2.24	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
105	36	942.0	2.90	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
106	30	799.0	2.75	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
107	30	560.0	1.80	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
108	24	862.0	2.97	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
109	30	399.0	1.10	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
110	310	359.0	0.99	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
111	38	306.0	1.06	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
112	38	629.0	2.02	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
113	40	265.0	0.73	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
114	25	890.0	2.86	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138

TABLE D-6. (continued)

SUBCATCH-MENT NO.	GUTTER OR INLET	WIDTH (FT)	AREA (AC)	PERCENT IMPERV.	SLOPE (FT/FT)	RESISTANCE FACTOR		SURFACE STORAGE(IN)		INFILTRATION RATE(IN/HR)		DECAY RATE (1/SEC)
						IMPERV.	PERV.	IMPERV.	PERV.	MAXIMUM	MINIMUM	
224	243	287.0	1.32	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
225	244	134.0	0.37	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
226	245	299.0	1.10	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
227	246	78.0	0.18	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
228	247	280.0	1.03	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
229	249	310.0	1.27	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
230	251	607.0	2.09	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
231	248	171.0	0.59	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
232	250	415.0	1.43	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
233	246	596.0	2.07	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
234	244	548.0	1.76	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
235	231	843.0	4.84	29.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
236	236	655.0	4.51	29.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
237	356	545.0	3.38	47.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
244	222	632.0	3.63	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
245	233	419.0	1.25	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
246	233	1106.0	3.30	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
247	231	746.0	2.57	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
248	229	727.0	2.17	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
249	235	911.0	2.72	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
250	252	286.0	0.92	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
251	251	419.0	1.54	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
252	252	787.0	2.35	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
253	229	886.0	3.05	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
254	253	296.0	0.95	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
255	253	700.0	2.57	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
256	228	351.0	1.21	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
257	228	436.0	1.50	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
258	230	355.0	1.14	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
259	232	575.0	3.30	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
260	222	171.0	0.59	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
262	219	2110.0	9.69	45.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
264	352	224.0	0.77	46.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
268	221	338.0	0.62	47.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
269	350	436.0	1.50	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
270	221	389.0	1.25	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
271	220	599.0	2.75	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
272	248	685.0	2.20	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
273	220	719.0	3.30	47.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
275	211	312.0	1.36	74.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
276	223	566.0	2.86	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
277	225	993.0	2.28	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
278	225	774.0	2.31	44.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
279	226	789.0	3.08	44.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
280	225	278.0	0.51	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
281	227	1263.0	2.90	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
261	232	450.0	1.65	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
289	346	903.0	2.28	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
307	233	346.0	1.39	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
308	234	462.0	1.06	43.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
266	221	1190.0	6.83	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
210	264	1126.0	5.43	49.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
211	264	639.0	2.20	65.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
212	364	1162.0	4.00	43.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
213	362	549.0	2.90	62.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
214	362	1157.0	6.64	53.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
215	263	396.0	2.09	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
216	356	632.0	3.19	48.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
217	358	501.0	1.84	45.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
238	358	686.0	3.67	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
239	262	498.0	2.86	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
240	262	620.0	3.56	50.0	0.0200	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
241	260	435.0	2.50	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
242	261	611.0	3.49	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
243	261	364.0	1.17	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
263	251	457.0	0.84	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
265	261	802.0	1.84	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
290	346	243.0	0.55	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138
267	260	1214.0	6.13	50.0	0.0100	0.013	0.250	0.062	0.250	3.00	0.28	0.00138

TABLE D-7. PIPE ELEMENT DATA FOR COMBINED SEWER SYSTEM

SUBAREA III

Pipe Number	Slope	Length (Ft)	Tributary Area (Acres)	Surface Elevation (ft. MSL)
201	.0050	220	8.88	56.80
202	.0013	440	4.00	52.00
203	.0043	200	13.31	53.00
204	.0043	250	1.28	53.50
205	.0054	300	5.17	51.00
206	.0054	230	2.20	49.00
207	.0290	320	1.61	38.50
208	.0025	480	5.28	38.50
212	.0100	850	16.52	42.00
213	.0100	450	2.39	33.00
214	.0100	470	8.99	28.00
215	.0025	300	5.10	40.00
216	.0025	750	5.10	38.50
209	.0193	350	2.09	32.80
210	.0092	250	1.84	30.00
211	.0248	300	1.36	28.00
219	.0025	1250	9.69	37.50
		SUBAREA IV		
220	.0025	400	6.02	31.20
221	.0025	350	8.70	31.20
227	.0025	650	2.90	32.80
226	.0025	280	3.08	31.00
225	.0025	330	4.59	29.00
228	.0025	350	3.22	30.50
229	.0025	280	5.22	30.50
230	.0025	260	1.14	30.00
233	.0025	290	5.94	31.00
222	.0025	165	4.20	30.50
232	.0025	500	4.95	30.00
235	.0025	320	2.72	34.00
234	.0025	300	1.06	33.00
231	.0025	580	10.50	30.00
224	.0070	180	0	29.00
223	.0030	400	2.86	28.00
236	.0025	540	4.51	33.00
238	.0040	250	5.84	35.00
246	.0020	250	2.20	36.40
249	.0025	340	3.66	33.20
251	.0025	300	3.63	34.00
253	.0025	300	3.53	35.00
252	.0025	250	3.27	34.00
250	.0025	250	1.43	32.00
248	.0012	250	0.59	33.20
247	.0010	360	3.60	36.40
245	.0010	320	3.38	38.00
244	.0020	250	2.13	38.00
243	.0010	320	3.78	36.00
242	.0042	320	4.63	34.00
241	.0050	300	4.88	32.00
240	.0050	300	5.22	33.00
239	.0050	300	2.02	35.00
237	.0040	400	5.43	33.00

SUBAREA V

Pipe Number	Slope	Length (Ft)	Tributary Area (Acres)	Surface Elevation (ft. MSL)
260	.0025	480	8.63	38.00
261	.0025	540	7.78	38.00
262	.0025	490	6.42	37.50
263	.0025	350	2.09	37.50
264	.0025	360	7.63	32.00
		TRUNK SEWER		
301	.0025	130.0	0	71.69
303	.0025	250.0	1.87	70.55
305	.0025	250.0	0	69.89
307	.0025	270.0	0.66	69.09
309	.0025	290.0	0.81	69.44
311	.0025	300.0	0.99	67.95
313	.0025	800.0	1.10	61.30
315	.0025	420.0	4.07	62.20
317	.0025	390.0	1.69	59.00
319	.0090	330.0	6.49	57.20
321	.0031	240.0	1.76	56.00
323	.0017	250.0	1.84	56.50
325	.0020	260.0	0	54.50
327	.0099	320.0	0	51.80
329	.0048	320.0	1.28	49.50
331	.0019	390.0	4.40	46.80
333	.0133	390.0	9.42	44.10
335	.0089	350.0	1.69	42.00
337	.0091	340.0	5.65	39.60
339	.0034	340.0	1.50	38.10
341	.0019	220.0	2.24	36.50
343	.0019	430.0	8.20	36.00
345	.0019	320.0	4.10	32.00
347	.0004	870.0	2.83	31.20
349	.0004	430.0	2.20	37.00
351	.0004	220.0	1.50	37.50
353	.0013	710.0	0.77	38.00
357	.0030	340.0	3.38	30.50
359	.0030	470.0	8.70	37.50
361	.0030	900.0	0	32.00
363	.0030	750.0	13.54	30.00

TABLE D-7. (continued)

SUBAREA I

Pipe Number	Slope	Length (Ft)	Tributary Area (Acres)	Surface Elevation (ft. MSL)
1	.0025	280	3.71	73.24
3	.0032	240	3.60	72.49
4	.0025	280	4.66	72.49
7	.0098	220	1.65	77.62
8	.0098	420	3.05	73.30
10	.0098	300	3.78	71.69
13	.0025	240	3.23	77.00
14	.0025	570	3.38	70.55
15	.0092	310	3.05	70.55
20	.0018	310	3.01	69.89
19	.0025	570	6.42	69.89
24	.0025	570	6.79	69.09
25	.0053	310	2.86	69.09
29	.0025	230	2.75	76.00
30	.0025	300	5.65	74.81
33	.0025	220	3.56	76.00
31	.0025	235	3.45	74.81
35	.0212	300	0	68.44
42	.0071	275	2.82	72.52
40	.0071	260	3.12	70.63
38	.0071	280	3.08	68.95
36	.0013	620	5.14	68.44
47	.0091	480	4.58	67.95
49	.0025	600	3.56	63.50
56	.0025	360	4.15	62.00
52	.0025	400	6.26	64.00
54	.0025	280	1.87	63.00
55	.0025	240	2.02	62.00
114	.0025	220	0	61.30
117	.0025	370	4.11	63.00
59	.0025	680	4.15	62.00
72	.0025	420	4.95	71.00
74	.0025	260	4.98	72.00
75	.0025	540	3.12	71.00
67	.0025	290	3.47	68.50
105	.0025	150	0	62.00
109	.0025	150	0	61.30
129	.0025	690	1.60	73.00
77	.0065	690	0	72.00
76	.0065	220	1.36	71.00
71	.0065	300	1.14	69.00
69	.0065	300	5.50	68.50
64	.0038	650	7.16	68.00
62	.0036	290	2.28	65.00
60	.0063	290	4.44	62.00
58	.0184	270	1.28	62.00

SUBAREA II

Pipe Number	Slope	Length (Ft)	Tributary Area (Acres)	Surface Elevation (ft. MSL)
82	.0025	170	4.51	57.70
101	.0025	550	6.69	64.30
99	.0075	280	3.26	62.00
98	.0025	320	4.26	62.00
95	.0062	250	0.92	61.00
94	.0025	860	6.83	62.00
92	.0141	230	2.49	59.40
91	.0025	860	6.79	59.40
189	.0104	230	2.31	56.70
187	.0425	150	4.36	56.50
85	.0025	660	7.56	56.50
104	.0217	270	3.63	54.50
103	.0025	610	4.55	54.50
108	.0025	640	7.68	51.80
112	.0025	570	6.68	49.50
119	.0025	390	5.02	56.00
78	.0025	270	1.36	55.00
80	.0025	300	2.54	54.60
83	.0055	160	0	55.00
121	.0025	350	4.18	56.00
123	.0025	250	2.17	55.00
125	.0025	450	3.52	55.00
126	.0054	200	0	54.90
84	.0059	200	4.44	54.80
128	.0233	580	8.55	44.10
148	.0025	650	3.27	69.00
149	.0388	250	0.88	60.10
147	.0413	250	2.83	50.50
146	.0118	380	7.68	48.30
143	.0025	360	6.35	46.20
141	.0058	350	7.07	44.10
139	.0026	310	5.98	43.10
138	.0025	250	1.46	43.10
135	.0085	150	2.39	42.80
87	.0085	150	0.73	42.20
89	.0051	220	3.01	41.00
133	.0051	310	1.39	39.60

TABLE D-8. SUBCATCHMENT WATER QUALITY INPUT DATA FOR SWMM

SUBAREA NUMBER	LAND USE CLASS.	TOTAL GUTTER LENGTH*10**2 FT.	NUMBER OF CATCHBASINS
1	2	10.00	0.0
2	2	4.00	0.0
3	2	5.00	0.0
4	2	6.00	0.0
5	2	5.00	0.0
6	2	8.00	0.0
7	1	4.00	0.0
8	1	6.00	0.0
9	1	4.00	0.0
10	1	5.00	0.0
11	1	3.00	0.0
12	1	6.00	0.0
13	1	15.00	0.0
14	1	12.00	0.0
15	1	8.00	0.0
16	1	7.00	0.0
17	1	7.00	0.0
18	1	7.00	0.0
19	1	8.00	0.0
20	1	11.00	0.0
21	1	8.00	0.0
22	1	11.00	0.0
23	1	8.00	0.0
24	1	6.00	0.0
25	1	11.00	0.0
26	1	14.00	0.0
27	1	6.00	0.0
28	1	13.00	0.0
29	1	6.00	0.0
30	1	10.00	0.0
31	1	6.00	0.0
32	1	10.00	0.0
33	1	7.00	0.0
34	1	16.00	0.0
37	1	20.00	0.0
38	1	16.00	0.0
39	1	14.00	0.0
40	1	3.00	0.0
41	1	5.00	0.0
42	1	8.00	0.0
43	1	9.00	0.0
44	1	5.50	0.0
54	1	13.00	0.0
55	1	4.00	0.0
56	1	7.00	0.0
57	1	4.50	0.0
58	1	27.00	0.0
59	1	27.00	0.0
60	1	15.00	0.0
61	1	6.00	0.0
62	1	5.00	0.0
63	1	8.00	0.0
64	1	6.00	0.0
65	1	7.00	0.0
74	1	6.50	0.0
165	1	8.00	0.0
164	1	4.00	0.0
163	1	5.00	0.0
162	1	6.00	0.0
161	1	3.50	0.0
160	1	13.00	0.0
159	1	3.00	0.0
158	1	7.00	0.0
157	1	3.00	0.0
156	1	5.00	0.0
155	1	17.00	0.0
154	1	6.50	0.0
153	1	9.00	0.0
152	1	7.00	0.0
151	1	8.00	0.0
150	1	7.50	0.0
149	1	5.50	0.0
148	1	7.50	0.0
147	1	15.00	0.0
146	1	7.50	0.0
145	1	8.00	0.0
144	1	5.00	0.0
143	1	9.00	0.0
142	1	6.00	0.0
141	1	13.50	0.0
171	1	11.00	0.0
35	1	6.00	0.0
36	1	8.00	0.0
45	1	9.00	0.0
46	1	5.00	0.0
47	1	3.00	0.0
48	1	6.00	0.0
49	1	4.50	0.0
50	1	9.50	0.0
51	1	8.00	0.0
52	1	7.50	0.0
53	1	12.00	0.0
66	1	8.00	0.0

TABLE D-8. (continued)

SUBAREA NUMBER	LAND USE CLASS.	TOTAL GUTTER LENGTH*10**2 FT.	NUMBER OF CATCHBASINS
67	1	6.50	0.0
68	1	6.00	0.0
69	1	6.00	0.0
70	1	10.50	0.0
71	1	11.50	0.0
72	1	14.00	0.0
73	1	13.50	0.0
75	1	11.00	0.0
76	1	17.00	0.0
77	1	5.00	0.0
78	1	7.00	0.0
79	1	7.00	0.0
80	1	7.00	0.0
81	1	6.00	0.0
82	1	6.00	0.0
83	1	6.00	0.0
84	1	6.00	0.0
85	1	5.00	0.0
86	1	11.00	0.0
87	1	24.00	0.0
88	1	6.00	0.0
89	1	11.50	0.0
90	1	9.00	0.0
91	1	5.00	0.0
92	1	15.50	0.0
93	1	25.00	0.0
94	1	6.50	0.0
95	1	9.50	0.0
96	1	10.00	0.0
97	1	11.00	0.0
98	1	4.50	0.0
99	1	5.00	0.0
100	1	6.00	0.0
101	1	9.50	0.0
102	1	5.00	0.0
103	1	7.00	0.0
104	1	7.50	0.0
105	1	13.00	0.0
106	1	12.50	0.0
107	1	8.00	0.0
108	1	12.00	0.0
109	1	5.50	0.0
110	1	6.00	0.0
111	1	5.50	0.0
112	1	8.00	0.0
113	1	5.50	0.0
114	1	12.50	0.0
115	1	5.00	0.0
116	1	12.50	0.0
117	1	5.00	0.0
118	1	5.50	0.0
119	1	12.00	0.0
120	1	5.50	0.0
121	1	12.50	0.0
122	1	5.00	0.0
123	1	8.50	0.0
124	1	9.00	0.0
125	1	5.50	0.0
126	1	12.50	0.0
127	1	12.50	0.0
128	1	5.00	0.0
129	1	5.50	0.0
130	1	8.00	0.0
131	1	5.50	0.0
132	1	9.00	0.0
133	1	7.50	0.0
134	1	4.50	0.0
135	1	8.00	0.0
136	1	5.50	0.0
137	1	12.50	0.0
138	1	17.50	0.0
139	1	4.50	0.0
140	1	18.00	0.0
172	1	2.00	0.0
166	1	16.00	0.0
170	1	16.50	0.0
310	3	16.00	0.0
309	3	6.00	0.0
306	3	6.00	0.0
283	1	16.00	0.0
284	1	10.00	0.0
282	1	9.00	0.0
286	2	5.00	0.0
285	2	7.00	0.0
294	4	6.00	0.0
295	2	5.00	0.0
296	2	6.00	0.0
300	2	8.00	0.0
298	2	4.00	0.0
299	2	4.00	0.0
301	2	12.00	0.0
297	2	17.00	0.0
293	2	6.00	0.0

TABLE D-8. (continued)

SUBAREA NUMBER	LAND USE CLASS.	TOTAL GUTTER LENGTH*10**2 FT.	NUMBER OF CATCHBASINS
274	3	6.00	0.0
287	2	20.00	0.0
288	3	9.00	0.0
302	3	9.00	0.0
303	4	17.00	0.0
304	4	17.00	0.0
305	4	6.00	0.0
291	4	17.00	0.0
292	3	9.00	0.0
201	1	9.00	0.0
202	1	9.00	0.0
203	1	9.00	0.0
204	1	9.00	0.0
205	1	9.00	0.0
206	1	10.00	0.0
207	1	10.00	0.0
208	1	10.00	0.0
209	1	15.00	0.0
218	5	10.00	0.0
219	5	6.00	0.0
220	5	6.00	0.0
221	5	6.00	0.0
222	5	7.00	0.0
223	1	10.00	0.0
224	1	6.00	0.0
225	1	3.00	0.0
226	1	6.00	0.0
227	1	2.00	0.0
228	1	6.00	0.0
229	1	6.00	0.0
230	1	9.00	0.0
231	1	4.00	0.0
232	1	6.00	0.0
233	1	7.00	0.0
234	1	7.00	0.0
235	5	13.00	0.0
236	5	12.00	0.0
237	1	11.00	0.0
244	1	11.00	0.0
245	1	5.00	0.0
246	1	13.00	0.0
247	1	11.00	0.0
248	1	11.00	0.0
249	1	11.00	0.0
250	1	6.00	0.0
251	1	6.00	0.0
252	1	9.00	0.0
253	1	12.00	0.0
254	1	5.00	0.0
255	1	9.00	0.0
256	1	5.00	0.0
257	1	7.00	0.0
258	1	5.00	0.0
259	1	10.00	0.0
260	1	3.00	0.0
262	1	25.00	0.0
264	1	4.00	0.0
268	2	4.00	0.0
269	1	8.00	0.0
270	1	6.00	0.0
271	1	9.00	0.0
272	1	8.00	0.0
273	1	9.00	0.0
275	3	6.00	0.0
276	1	13.00	0.0
277	1	12.00	0.0
278	1	11.00	0.0
279	1	12.00	0.0
280	1	4.00	0.0
281	1	13.00	0.0
261	1	5.00	0.0
289	1	9.00	0.0
307	1	6.00	0.0
308	1	6.00	0.0
266	2	17.00	0.0
210	2	16.00	0.0
211	1	8.00	0.0
212	2	13.00	0.0
213	2	12.00	0.0
214	2	18.00	0.0
215	2	8.00	0.0
216	2	9.00	0.0
217	1	7.00	0.0
238	2	11.00	0.0
239	2	10.00	0.0
240	2	9.00	0.0
241	2	10.00	0.0
242	2	10.00	0.0
243	2	9.00	0.0
263	2	8.00	0.0
265	2	9.00	0.0
290	3	3.00	0.0
267	2	16.00	0.0

than each separately. The pervious and impervious areas of each subarea in District A were computed. The data on curb length to develop criteria as to dust and dirt accumulation is included in TABLE D-9 for each major subarea. The data were developed from an integration of subcatchment quality input parameters shown in TABLE D-8. TABLE D-10 summarizes the District A land use data which are used as direct input to the STORM program. The curb length in single family residential is the greatest among all types of land uses because, in Elizabeth, each single family lot is quite small, generally 0.08 hectares (0.20 acres) or less, and the street arrangement is compact.

TABLE D-9. SUBAREA CURB LENGTH (FEET) VERSUS LAND USE, DISTRICT A

Subarea	Single Family	Multiple Family	Commercial	Industrial	Parks
I	77,000	-	-	-	-
II	67,300	2,480	120	-	-
III	3,100	11,920	6,980	3,900	-
IV	44,385	5,540	1,810	-	3,685
V	1,795	15,585	920	-	-
Total	193,680	35,525	9,830	3,900	3,685

Note: 1 foot = 0.3048 meters

TABLE D-10. SUMMARY OF DRAINAGE DISTRICT A LAND USE DATA

Land Use	% of Area	% Impervious	Curb Length (ft/acre)
Single Family	71.7	43	413.
Multiple Family	18.2	50	298.
Commercial	5.3	80	283.
Industrial	2.8	80	216.
Open Space	2.0	19	296.

1 ft/acre = .753 m/hectare

The depression storage capacity as required by the STORM program is a single parameter average over the entire drainage basin being considered. With 47 percent impervious area, 1.59 mm (0.0625 inches) depression storage capacity for impervious areas and 6.35 mm (0.25 inches) for pervious areas, the average depression storage capacity for District A is computed as 3.94 mm (0.155 inches). In STORM, no runoff from a watershed takes place if the total rainfall is less than available depression storage. STORM allows depression storage to recover to its maximum capacity at a constant rate to account for evaporation. The recovery rates are assumed constant for a given month but may vary from month to month. The recovery of depression storage is the mechanism in STORM to account for antecedent dry conditions and enables the model to process continuous rainfall data.

The recovery rates are computed using the Meyer formula (40) with the following expression:

$$E = C(e_s - e_a)(1 + \frac{w}{10}) \qquad \text{(D-1)}$$

where:

E = the rate of evaporation in inches per month,

e_s = the saturation vapor pressure in inches of mercury (Hg) corresponding to monthly mean air temperature,

e_a = the vapor pressure of air in inches of Hg, based upon monthly mean air temperature and relative humidity,

W = the monthly mean wind speed in miles per hour obtained at 30 feet above general level of surrounding roofs of building,

C = a coefficient depending on various factors affecting evaporation and a value of 0.15 is suggested for small bodies of shallow waters such as wet soil surfaces and small puddles.

The monthly variation in evaporation rates for the study area were computed and are shown in TABLE D-11. The inputs to the Meyer formula for computing the rate of evaporation are also shown. The inputs were prepared using the climatological data recorded at Newark International Airport during the period of 1963-1974. The computed evaporation rates agree with the average annual lake evaporation in the New York area (41). Since the evaporation rates affect the available depression storage prior to a rainfall event, they have a more pronounced effect on runoff from low intensity rainfall than from high intensity rainfall of the same duration.

TABLE D-11. MONTHLY EVAPORATION RATE

Month	Mean Air Temperature (^oF)	Mean Relative Humidity (%)	Mean Wind Speed (mph)	Evaporation Rate (inches/day)
January	31.4	65.3	11.3	.065
February	31.1	59.3	12.2	.078
March	41.5	63.3	12.1	.105
April	51.9	57.0	11.6	.186
May	61.9	60.7	10.5	.232
June	71.9	61.5	9.6	.302
July	77.0	61.1	9.2	.322
August	76.0	66.7	9.2	.288
September	68.5	66.9	9.6	.233
October	57.4	66.2	9.6	.159
November	46.9	66.9	10.5	.112
December	36.6	68.0	11.1	.078

Note: 1 mph= 1.61 km per hour
1 inch/day = 25.4 mm/day

Pollutant washoff coefficient K is determined by the equation:

$$P_o - P = P_o(1-e^{-kr\Delta t}) \qquad \text{(D-2)}$$

where:

$P_o - P$ = pounds of washoff in a time interval, Δt,
P^o = pounds of pollutant in the street at the start of rainfall,
P^o = pounds of pollutant remaining at the end of the time interval,
Δt = the time interval, and
r = runoff rate in inches per hour.

In SWMM, K is internally set equal to a value of 4.6, based on the assumption that a uniform runoff rate of 12.7 mm (0.5 inches) per hour would wash away 90 percent of the pollutant in one hour. For consistent comparision of the results obtained with STORM and SWMM, the same washoff coefficient is used for the STORM. There are studies in which other K values are used (42).

Data for dry-weather flow quantity and quality required for simulation of combined sewage overflow are discussed elsewhere.

AREA AND LAND USES FOR OTHER DISTRICTS

The City has been divided into 25 drainage districts. These districts, lettered A through Y, are shown in Figure D-14, which also shows the Easterly and Westerly Interceptors and their corresponding inlet and sewer pipe elements.

Only composite data were required for drainage districts other than District A. TABLE D-12 presents for each district the area, the various kinds of land use, and the percent imperviousness. The areas of the planned land use were taken from the approved master plan (43). Where planned ultimate land use differed from the existing use, the use resulting in the greater population was adopted. The percent imperviousness in each district was the weighted average based on the percent imperviousness for various land uses as shown in TABLE D-10. Two-family houses were considered as part of the multiple family residential area. District A is the largest district in the City and includes all land uses and, therefore, permits developing relevant parameters for City-wide application.

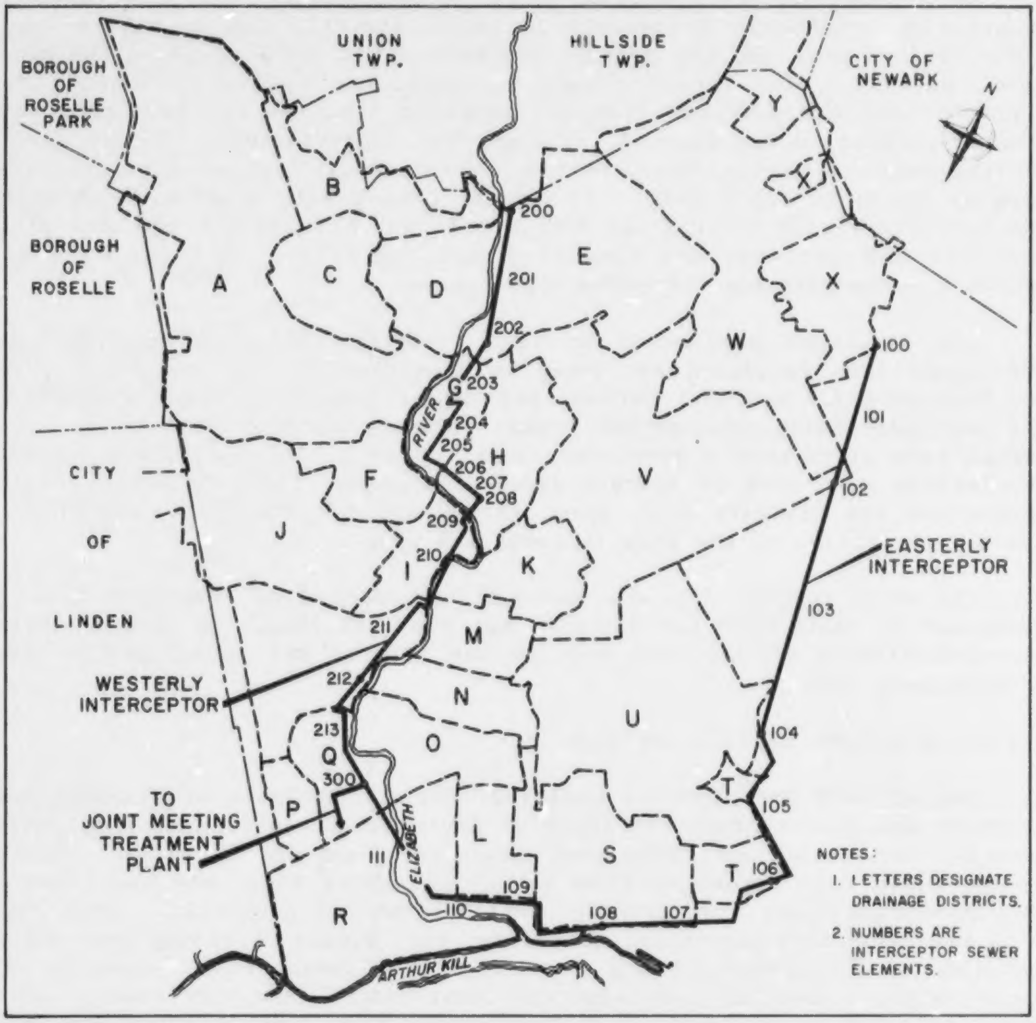
The developed population of each district are shown in TABLE D-13. The populations estimated are based on densities of 55.8 and 121.3 persons per hectare (22.6 and 49.1 persons per acres) respectively for single-family and multiple-family residential areas. These densities were obtained from census data at presently developed areas for each land use. These estimated populations were used to compute dry-weather flow. The estimated developed population for District A is about 16,500 and for the City, 124,000. The present population of the City is about 115,000.

The curb lengths for the various land uses were developed from data presented in TABLE D-10 for District A. The curb length is used to estimate the accumulation of dust and dirt on the streets and to define the amount of pollutant washoff.

DRY-WEATHER FLOW QUANTITY AND QUALITY

Use of SWMM and STORM as modified, for the analysis of combined sewage quantity and quality requires input of dry-weather flows, including domestic sewage, commercial and industrial discharges and infiltration. Domestic sewage flows were estimated from daily per capita water use and commercial and industrial flows from average contributions per area unit. Both commercial and industrial wastewater quantities vary widely depending upon the type and size of the commercial and industrial development (44). Quality components in the commercial and industrial wastewaters also vary widely (45,46). Both the quantity and quality of dry-weather flow vary with the hour of the day and the day of the week. While dry-weather flows make a significant contribution to the quality of combined sewage, their average quantity is small, generally less than 0.2 percent of the sewer design flow computed, using SWMM, with the 5-year return interval, intermediate pattern, synthetic storm hyetograph (Figure D-5).

With the assumed per capita contribution of 379 liters (100 gallons) per day from all sources, the average daily wastewater quantity from District A is 6,245 m^3 (1.65 million gallons). The average daily concentrations of pollutants in the wastewaters adopted are shown in TABLE D-14.


UNION TWP.
HILLSIDE TWP.
CITY OF NEWARK
BOROUGH OF ROSELLE PARK
BOROUGH OF ROSELLE
CITY OF LINDEN
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110
111
RIVER
ELIZABETH
ARTHUR KILL
EASTERLY INTERCEPTOR
WESTERLY INTERCEPTOR
TO JOINT MEETING TREATMENT PLANT
NOTES:
1. LETTERS DESIGNATE DRAINAGE DISTRICTS.
2. NUMBERS ARE INTERCEPTOR SEWER ELEMENTS.

Figure D-14. Definition of drainage districts and interceptor system in Elizabeth

TABLE D-12. SUMMARY OF LAND USE IN THE CITY OF ELIZABETH

District	Area (acres)	Land Use (%)					District Percent Impervi-ousness
		Single Family	Multiple Family	Commercial	Industrial	Park	
A	655	72	18	5	3	2	47
B	112	90	10	0	0	0	44
C	111	20	55	25	0	0	56
D	117	10	60	30	0	0	58
E	229	35	35	30	0	0	57
F	88	0	70	30	0	0	59
G	34	0	0	100	0	0	80
H	122	0	25	75	0	0	73
I	70	0	85	15	0	0	55
J	420	10	70	15	5	0	55
K	65	0	60	40	0	0	62
L	66	0	50	10	40	0	65
M	67	0	90	5	0	5	50
N	79	0	100	0	0	0	50
O	103	5	0	0	50	45	49
P	37	0	50	50	0	0	65
Q	83	10	40	25	25	0	64
R	303	0	0	0	100	0	80
S	207	0	75	5	15	5	54
T	62	0	50	0	50	0	65
U	331	0	50	25	25	0	65
V	440	0	40	27	30	3	66
W	415	0	90	10	0	0	53
X	138	15	65	0	0	20	43
Y	56	0	100	0	0	0	50
City	4410	17.3	47.5	16.3	16.3	2.6	58

NOTE: 1 acre = 0.405 hectares

Acres of Land Use indicated are based on approved Master Plan of the City (43)

TABLE D-13. POPULATION AT SATURATION DEVELOPMENT

District	Area (Acres)	Single* Family	Multiple** Family	Total
A	655	10,658	5,789	16,447
B	112	2,278	550	2,828
C	111	502	2,998	3,500
D	117	264	3,447	3,711
E	229	1,811	3,935	5,746
F	88	0	3,025	3,025
G	34	0	0	0
H	122	0	1,498	1,498
I	70	0	2,921	2,921
J	420	949	18,560	19,509
K	65	0	1,915	1,915
L	66	0	1,620	1,620
M	67	0	2,961	2,961
N	79	0	3,879	3,879
O	103	116	0	116
P	37	0	908	908
Q	83	188	750	938
R	303	0	0	0
S	207	0	7,623	7,623
T	62	0	1,522	1,522
U	331	0	8,126	8,126
V	440	0	8,642	8,642
W	415	0	18,339	18,339
X	138	468	4,404	4,872
Y	56	0	2,750	2,750
Total	4,410	17,234	106,162	123,396

* Based on 22.6 persons per acre (55.8 persons per hectare)

** Based on 49.1 persons per acre (121.3 persons per hectare)

Population Densities for indicated land uses are derived using census data from presently developed areas.

TABLE D-14. AVERAGE DAILY POLLUTANT CONCENTRATION OF DOMESTIC WASTEWATER

Pollutant	Concentration
Suspended solids	240 mg/l
BOD	200 mg/l
Settleable solid	6 mg/l
Total nitrogen	30 mg/l
Total phosphate	10 mg/l
Total coliform	5.27×10^7 MPN/100ml

These pollutant concentrations are classified as "medium" according to an EPA study report (45). Diurnal variations of sanitary wastewater quantity and quality are shown in TABLE D-15. These variations are based on experience in cities with population and urbanization comparable to Elizabeth. The peak hourly rate occurs at 10 a.m. and equals 1.45 times the daily average flow rate. For District A, the peak hourly wastewater flow rate is 0.105 m^3/sec (2.4 mgd). Possible variations in quality and quantity of wastes by day of the week were not considered.

The hourly DWF pollutant loadings for SS, BOD, and total coliform in TABLE D-15 are based on the daily average concentrations shown in TABLE D-14. It was assumed that the same concentration ratio variation applies to all pollutants. During the preparation of input data to STORM, pollutant concentration ratios in TABLE D-15 were used as pollutant mass loading. This artifically increased the pollutant mass loading in the early hours of the day and decreased it in the later hours. As a result, the amount of untreated sanitary sewage in the overflow may either increase or decrease for individual storm events, depending upon the time of the day the rainfall starts. On the average, it may increase the computed amounts of annual pollutants discharged by about 10 percent. This deviation in computing annual amount of pollutants in the overflow would be the same for all alternatives tested and would not change the relative proportions of pollutants discharged which were used as a measure for determining the desirable systems. Based on the assumed SS contribution from commercial and industrial areas to be 0.06 and 0.08 kg per hectare (0.33 and 0.44 pounds per acre) per day respectively, the total non-domestic portion of SS in the dry-weather flow would be about 9 kg (20 pounds), or less than one percent of the assumed wastewater loading. The diurnal variations shown in TABLE D-15 are DWF input data to STORM.

In SWMM, consideration of DWF is made in the TRANSPORT Block. Thus, for District A, DWF enters the trunk sewers directly. The district was divided into 25 subareas. TABLE D-16 presents, for each subarea, the inlet manhole, area, predominant land use, population density, average daily flow and daily SS and BOD loading. Other than daily pollutant loadings,

TABLE D-15. DIURNAL VARIATION OF DOMESTIC WASTEWATER QUANTITY AND QUALITY FOR DISTRICT A

Hour of the day	Flow Ratio*	Flow Rate (mgd)	Concentration Ratio*	SS (lbs/hr)	BOD (lbs/hr)	Total Coliform MPN x 10^{15}/hr
1	.9	1.49	.90	123	103	.124
2	.7	1.16	.70	96	80	.096
3	.5	.83	.60	82	68	.083
4	.35	.58	.50	69	57	.069
5	.30	.50	.45	62	51	.062
6	.30	.50	.40	55	46	.055
7	.40	.66	.45	62	51	.062
8	.85	1.40	.50	69	57	.069
9	1.35	2.23	.60	82	68	.083
10	1.45	2.40	.70	96	80	.096
11	1.35	2.23	.85	116	97	.117
12	1.30	2.15	1.15	158	131	.158
13	1.25	2.06	1.25	171	143	.172
14	1.20	1.98	1.50	206	171	.206
15	1.15	1.90	1.55	212	177	.213
16	1.05	1.73	1.55	212	177	.213
17	1.05	1.73	1.50	206	171	.206
18	1.15	1.90	1.40	172	160	.193
19	1.25	2.06	1.35	185	154	.186
20	1.35	2.23	1.30	178	148	.179
21	1.35	2.23	1.25	171	143	.172
22	1.25	2.06	1.20	164	137	.165
23	1.15	1.90	1.20	164	137	.165
24	1.05	1.73	1.15	158	131	.158
Daily Average =		1.65	Daily Total =	3289	2738	3.3

* Ratios are with respect to the corresponding average values.

Note: 1 mgd = 0.0438 m^3/sec
1 lb = 0.453 kg

TABLE D-16. SWMM INPUT DATA FOR DOMESTIC WASTEWATER COMPUTATION, DISTRICT A

Subarea No.	Inlet Manhole	Area (Acres)	Land* Use	Population Density (persons per acre)	Average Daily Flow (cfs)	Average Daily Loading SS (lbs/day)	Average Daily Loading BOD (lbs/day)
1	360	14.53	2	37.0	.083	107.4	89.5
2	358	31.56	2	47.8	.233	301.5	251.2
3	356	8.70	2	43.1	.058	75.0	62.5
4	352	67.10	1	22.6	.237	306.6	255.5
5	352	135.35	1	23.4	.469	641.8	534.8
6	353	10.46	1	22.6	.036	46.6	38.8
7	350	21.32	1	30.7	.101	130.7	108.9
8	244	4.10	3	15.7	.010	12.9	10.8
9	342	8.20	3	22.6	.028	36.2	30.2
10	338	46.78	1	22.6	.164	212.2	176.8
11	334	37.32	1	30.6	.177	229.0	190.8
12	330	17.75	1	22.6	.062	80.2	66.8
13	328	10.46	1	22.6	.037	47.8	39.9
14	326	11.48	1	22.6	.040	51.8	43.1
15	324	39.14	1	22.6	.137	177.3	147.7
16	316	17.64	1	22.6	.062	80.2	66.8
17	316	54.08	1	22.6	.181	234.2	195.2
18	314	24.35	1	22.6	.085	110.0	91.6
19	312	8.65	1	22.6	.030	38.8	32.3
20	310	32.69	1	22.6	.115	148.8	124.0
21	308	10.64	1	22.6	.037	42.9	39.9
22	306	10.24	1	22.6	.036	46.6	38.8
23	304	10.32	1	22.6	.036	46.6	38.8
24	302	10.35	1	22.6	.036	46.6	38.8
25	300	11.97	1	22.6	.043	55.6	46.4
					2.56	3307.3	2760.

* Predominant Land Use
1 - Single Family Residential
2 - Multiple Family Residential
3 - Commercial

Note: 1 person/acre = 2.471 persons/hectare
1 cfs = 0.0283 m^3/sec
1 lb/day = 0.453 kg/day

which are computed internally in the program, additional data are required for the computation of combined sewage. The slight differences for daily SS and BOD loadings between TABLE D-15 (for STORM) and TABLE D-16 (for SWMM) are due mainly to the significant number contained in the SWMM print out and are inconsequential. The same diurnal quantity and quality variations shown in TABLE D-15 were applied to SWMM. In SWMM computations, the simulations were assumed to begin at 10 a.m. when the hourly DWF quantity was greatest.

UNIT COST DATA

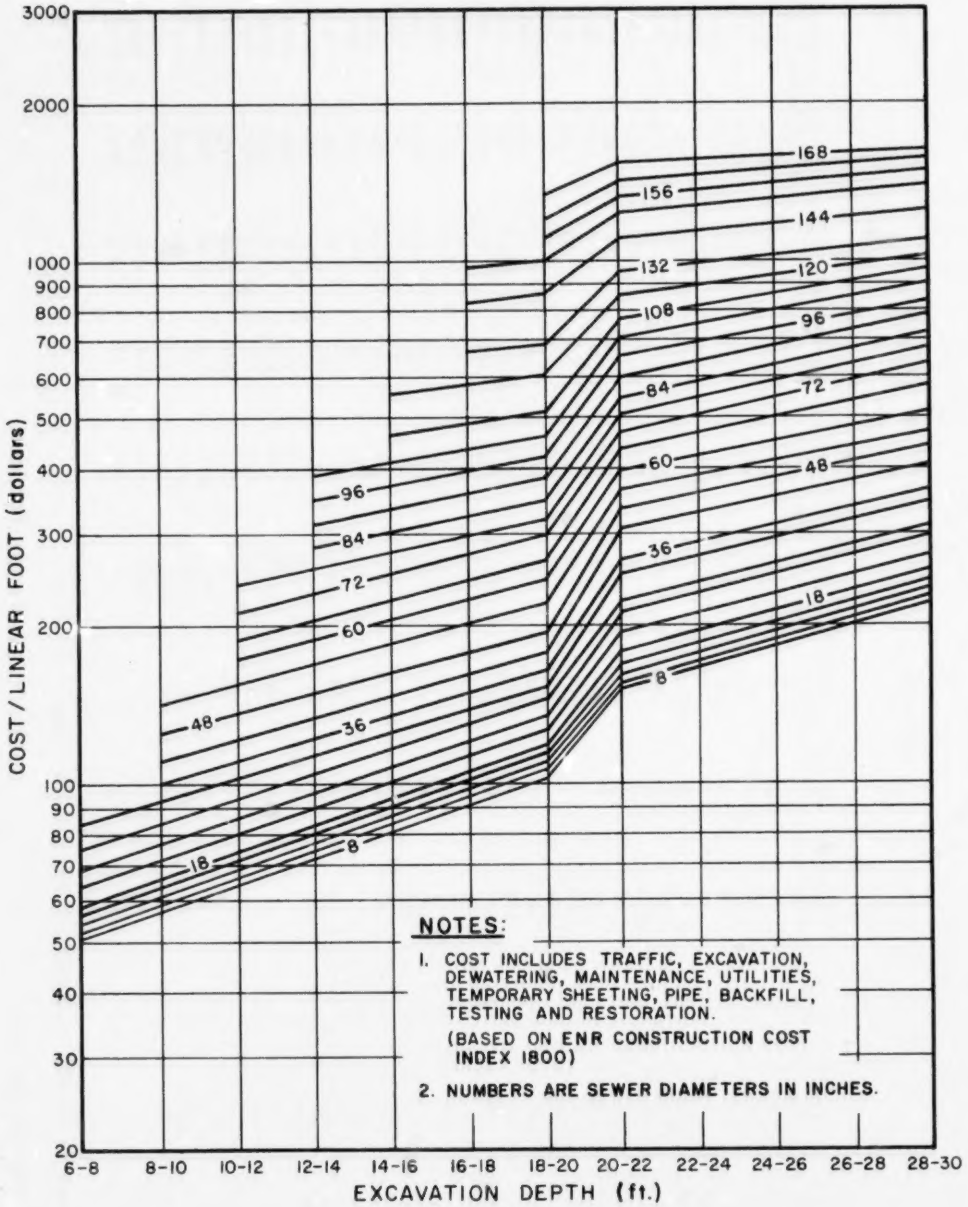
To compare the cost-effectiveness of the alternative sewer systems, cost data were developed for sewers, storage facilities, pumping stations and treatment works. These costs are affected by variables such as land, weather, subsurface conditions, groundwater, access, size of facilities, process involved, etc. The cost data used are applicable to work in Elizabeth, New Jersey and consolidate many factors that affect cost into a single parameter (such as costs per foot for each size of sewer for various depths of excavation, per gallon of storage capacity, per mgd of pumping capacity, etc.). These parameters are adequate for comparative purposes but would require refinement for definitive projects at specific sites.

Sewer Costs

Figure D-15 and TABLE D-17 present pipe cost per linear foot for various depths of trench developed from actual cost data. The unit cost includes the cost of pipe, pipe laying, excavation, dewatering, temporary tight sheeting and backfill, and is based on the ENR construction cost index of 1,800. Pipe sizes smaller than 0.457 m (18 inches) are used only for sanitary sewers. Circular pipe sewers were assumed. The unit price increases substantially at about a 6.1 m (20 feet) trench depth due to the additional bracing and sheeting required and greater excavation costs.

Storage Costs

Costs of storage facilities presently in operation have been summarized (18, 19). The cost of in-pipe storage, as used in Seattle, Washington, was estimated at about $0.06 per liter ($0.23 per gallon) of storage, mostly for control and monitoring systems and automated regulating stations in existing pipes. The cost of off-line storage depends upon the type and can include open lagoons, concrete covered reservoirs or deep tunnels. The open lagoon, such as the one used in Chippewa Falls, Wisconsin, costs about $.07 per liter ($0.26 per gallon) of storage. The cost range for covered concrete storage is from $0.13 per liter ($0.50 per gallon) for Humboldt Avenue, Milwaukee to $0.56 per liter ($2.12 per gallon) for a more recently built storage basin at Jamaica Bay, New York City. For the deep tunnel storage system in the City of Chicago, the cost was estimated as $0.06 per liter ($0.24 per gallon). The above costs are based on the ENR construction cost index of 2,000. In a sewerage improvement program recently developed for the City of Trenton, New Jersey (16), the cost of a sewage detention basin and appurtenances was estimated at about $0.07 per liter ($0.25 per gallon)



NOTES:

1. COST INCLUDES TRAFFIC, EXCAVATION, DEWATERING, MAINTENANCE, UTILITIES, TEMPORARY SHEETING, PIPE, BACKFILL, TESTING AND RESTORATION.

   (BASED ON ENR CONSTRUCTION COST INDEX 1800)

2. NUMBERS ARE SEWER DIAMETERS IN INCHES.

Figure D-15. Unit cost of sewers

TABLE D-17. UNIT COST OF SEWERS*

Pipe Diameter (ft)	Excavation Depth (feet) 6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	22-24	24-26	26-28	28-30	30-32
.67	51	58	66	75	85	96	108	151	166	183	200	220	240
.83	52	59	67	76	87	98	113	159	174	190	209	228	250
1.0	53	61	69	78	89	102	115	164	180	197	217	237	260
1.25	55	63	71	81	92	104	120	172	187	204	224	244	270
1.5	59	67	75	85	96	108	122	180	205	220	240	260	280
1.75	64	72	81	91	108	115	128	200	215	235	255	278	300
2.0	-	77	87	97	108	121	135	220	232	255	275	300	322
2.25	-	85	95	105	117	130	145	230	250	270	292	320	340
2.5	-	92	103	114	126	140	155	245	267	290	315	340	370
3.0	-	102	113	124	137	150	166	270	290	312	340	365	395
3.5	-	110	123	135	148	168	180	310	332	356	380	410	440
4.0	-	-	135	148	162	177	194	340	365	390	418	445	480
4.5	-	-	154	167	183	200	219	372	395	420	450	480	510
5.0	-	-	172	188	206	225	245	408	430	457	485	515	550
5.5	-	-	195	211	230	250	273	442	470	502	535	570	610
6.0	-	-	-	235	255	277	298	480	515	552	595	640	690
6.5	-	-	-	261	280	301	322	520	552	590	632	680	720
7.0	-	-	-	288	309	329	350	557	597	640	680	730	780
7.5	-	-	-	318	339	360	385	605	645	690	730	780	830
8.0	-	-	-	-	370	393	420	660	700	740	785	830	880
8.5	-	-	-	-	409	430	452	715	760	800	855	905	960
9.0	-	-	-	-	468	490	510	770	810	860	910	960	1020
9.5	-	-	-	-	520	540	550	815	860	910	960	1000	1050
10.0	-	-	-	-	-	590	612	850	900	950	1000	1010	1070
10.5	-	-	-	-	-	655	680	910	950	985	1035	1050	1100
11.0	-	-	-	-	-	710	740	960	1000	1010	1070	1100	1130
11.5	-	-	-	-	-	770	800	1000	1070	1100	1110	1180	1200
12.0	-	-	-	-	-	-	855	1100	1130	1190	1210	1260	1300
12.5	-	-	-	-	-	-	1000	1220	1280	1310	1340	1400	1420
13.0	-	-	-	-	-	-	1100	1310	1360	1400	1420	1480	1510
13.5	-	-	-	-	-	-	1205	1405	1450	1480	1510	1560	1600
14.0	-	-	-	-	-	-	-	1520	1560	1580	1600	1630	1680

* based on the ENR construction cost index of 1,800

Note: 1 foot = 0.3048 meter

of storage, based on the ENR construction cost index of 2,800. This cost included an improvement of the existing 64,345 m^3 (17 million gallons) detention basin, a 8.3 m^3/sec (190 mgd) capacity pumping station and over 610 m (2,000 feet) of inlet and outlet sewers.

For the purpose of this study it appears reasonable to assume, from the above data, a cost of \$0.08 per liter (\$0.30 per gallon) of storage. Higher storage costs, up to \$0.26 per liter (\$1.00 per gallon), were also considered to analyze the tradeoff between various amounts of storage facilities and varying collection sewer, interceptor, and treatment plant capacities for joint flood and pollution control where local conditions would result in higher storage construction costs.

Pumping Station Costs

Pumping is generally required to deliver to or return from storage wet weather flows. In very steep terrain, gravity flow into and out of the storage facility may be possible. The cost of the pumping station proposed for Trenton is estimated at about \$1,710,000 or an average of about \$206,000 per m^3/sec (\$9,000 per mgd) based on the ENR construction cost index of 2,800. This cost has been used in this study without adjustment for cost index difference.

Treatment Costs

The alternative treatment processes for stormwater runoff and combined sewage and their associated treatment costs were presented in Lager and Smith (18) and later summarized by Field and Lager (19). The cost data plotted in Figure D-16 are taken from this latter reference. Capital cost per mgd of treatment capacity appears to increase nearly linearly with BOD removal efficiency. The microstrainer, which is relatively economical but relatively inefficient in removing BOD, is effective in SS removal with a removal rate of 70 percent (19). Biological treatment processes are impractical for satellite plants because of the problem of maintaining an effective biomass with intermittent flows. However, it is practical and may offer benefit to treat combined sewage or urban runoff at a central plant.

Elizabeth has been allocated a peak treatment rate of 1.75 m^3/sec (40 mgd) at the Joint Meeting Plant (about two times average flow). The plant is being upgraded to provide secondary treatment. Hence, one approach investigated provided external storage near the treatment plant for flow equalization to make effective use of the allocated capacity and as a result, the treatment costs for the alternative sewer systems were practically the same. A second approach assumed varying peak treatment rates (up to 17.5 m^3/sec or 400 mgd) and appropriately modified storage quantities for flow modulation. The SWMM STORAGE/TREATMENT Block was used to estimate costs under this approach. All costs were adjusted to the ENR construction cost index of 1,800.

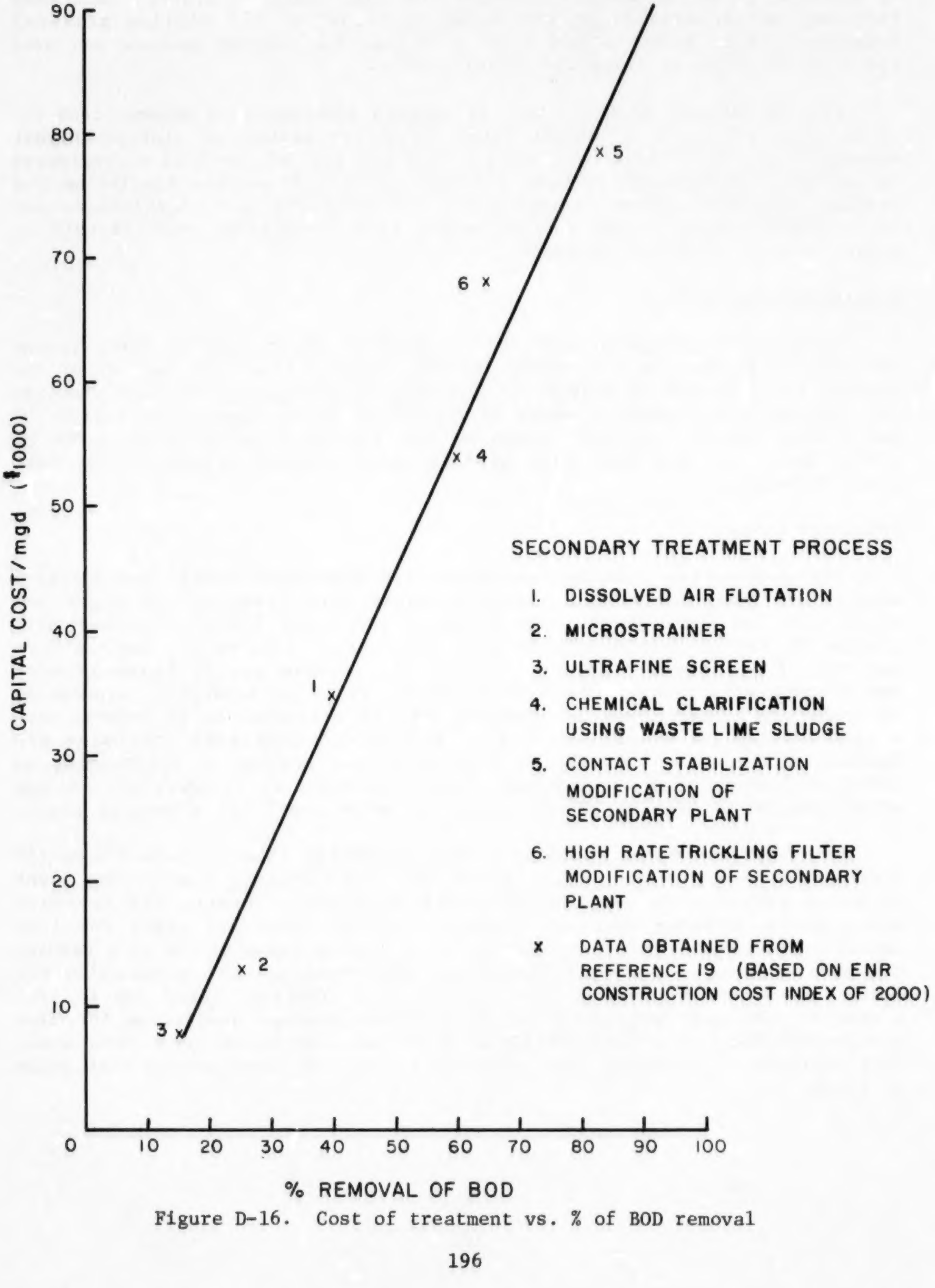


Figure D-16. Cost of treatment vs. % of BOD removal

APPENDIX E

CALIBRATION OF STORM RUNOFF COEFFICIENTS

Two runoff coefficients are specified in STORM: C_1 and C_2, respectively, for pervious and impervious areas. Infiltration losses are limited to pervious areas. Consequently, C_1 is variable while C_2 is essentially constant.

Infiltration in pervious areas is generally the most significant rainfall loss. Infiltration rates may vary from several inches per hour in extremely pervious deep sands to zero on tight clays. The infiltration rate is dependent on soil characteristics, the terrain, the rainfall rate, pattern and duration, and on the water content of the soil.

Because of the variability in the factors affecting the infiltration rate for a given area, C_1 is not susceptible to precise determination. It is higher for less frequent, more intense storms because infiltration and other losses have a proportionally smaller effect on runoff.

Runoff coefficients for STORM were calibrated using runoff data generated by SWMM in District A. These coefficients were applied to other drainage districts in the City to obtain the runoff volumes for synthetic or real storms. The following assumptions were made:

1. C_2 was set equal to 1.0, since there is no infiltration loss in impervious areas and the depression storage is accounted for separately.

2. Surface runoff data were generated by adapting the quantity portion of the SWMM RUNOFF Block to exclude sewer routing. This is consistent with STORM, in which the effect of sewer routing is not considered. All subcatchments in District A were used.

3. The depression storage capacities for pervious and impervious areas used in SWMM were 6.35 and 1.57 mm (0.25 and 0.062 inches) respectively. Twenty five percent of the impervious area was assumed to have no depression storage. The equivalent depression storage for District A was 3.94 mm (0.155 inches), based on 47 percent of the area being impervious.

4. The infiltration capacity curve shown in Figure D-5 was used in SWMM to account for the infiltration loss. Maximum and

minimum infiltration rates of 76.2 and 7.11 mm (3.0 and 0.28 inches) per hour, respectively, and a decay rate of 0.00138/sec was used. The antecedent conditions for rainfall events were such that the infiltration curve specified applied.

5. The typical rainfall event was assumed to have a 3-hour duration and an intermediate pattern similar to the 5-year synthetic storm pattern shown in Figure D-5.

C_1 was calibrated so that the 3-hour storm runoff volume computed with the computer program adapted from SWMM was the same as that computed with the STORM program.

Sensitivity runs were made using the adapted SWMM program to investigate the effect on the runoff volume of (a) using the hourly hyetograph as opposed to 2-minute hyetograph, and (b) the integration interval. These runs were made for the 5-year synthetic storm in District A. The results are presented in TABLE E-1.

TABLE E-1. SENSITIVITY OF HYETOGRAPH AND INTEGRATION INTERVAL ON RUNOFF VOLUME

Case No.	Hyetograph*	Integration Interval (min.)	5-Year Rainfall (inches)	Storm Runoff (inches)	Percent of Rainfall Runoff
1	2 - min	2	2.43	1.75	0.72
2	Hourly	2	2.43	1.69	0.70
3	Hourly	60	2.43	1.72	0.71

* See Figure D-5 for intermediate pattern storm hyetograph

Note: 1 inch = 25.4 mm

There is insignificant difference in runoff volume obtained by using the hourly hyetograph with a one-hour integration interval and that obtained by using the 2-minute hyetograph with a 2-minute integration interval. This finding does not apply to peak flow rates, however, where the hyetograph interval makes a substantial difference. The savings in computer costs in using hourly intervals to determine runoff volume are substantial being only 25 percent of those using 2-minute intervals. Hence, data generated with the adapted SWMM program for calibration purposes use the hourly hyetograph and integration interval.

The calibrated C_1 values are shown both in TABLE E-2 and Figure E-1 as a function of 3-hour rainfalls. C_1 increases with rainfall intensity.

TABLE E-2. CALIBRATION OF RUNOFF COEFFICIENT C_1

Case No.	3-hour Rainfall (inches)	Return Interval	Average Intensity (In/hour)	Runoff Volume (inches)	STORM C_1
1	4.86	>100-Year	1.62	4.15	0.78
2	2.43	5-Year	0.81	1.72	0.55
3	1.38	1-Year	0.46	0.76	0.30
4	0.61	1.3-Month	0.20	0.27	0.25

Note: District A data: Area = 655.1 Acres
Imperviousness = 47%

85 percent of the rainfalls during the period studied (1963-1974) totaled less than 0.6 inches

1 inch/hour = 25.4 mm/hour

The effect of storm duration on the runoff coefficient was also studied. Computer runs were made with assumed 6-hour storm duration maintaining the same average storm intensities as for the 3-hour storm. The increase in storm duration increases the total amount of infiltration. Hence, for the same average intensity rainfall, coefficients for longer duration storms are more affected than those for shorter duration storms. For instance, the C_1 value for a 3-hour storm of average intensity equal to 5.08 mm (0.20 inches) per hour is computed to be 0.23. For the same intensity storm with a 6-hour duration, C_1 equalled 0.11. For storms with average intensity equal to 20.6 mm (0.81 inches) per hour (5-year return frequency for the 3-hour storm), the difference in runoff coefficient C_1 was insignificant (0.55 versus 0.58). However, the return frequency of a 6-hour storm with an average intensity of 20.66 mm (0.81 inches) per hour is about 100 years or longer.

Considering that C_1 varies with the storm severity (depth, duration) and STORM assumes a constant runoff coefficient for the entire time span of

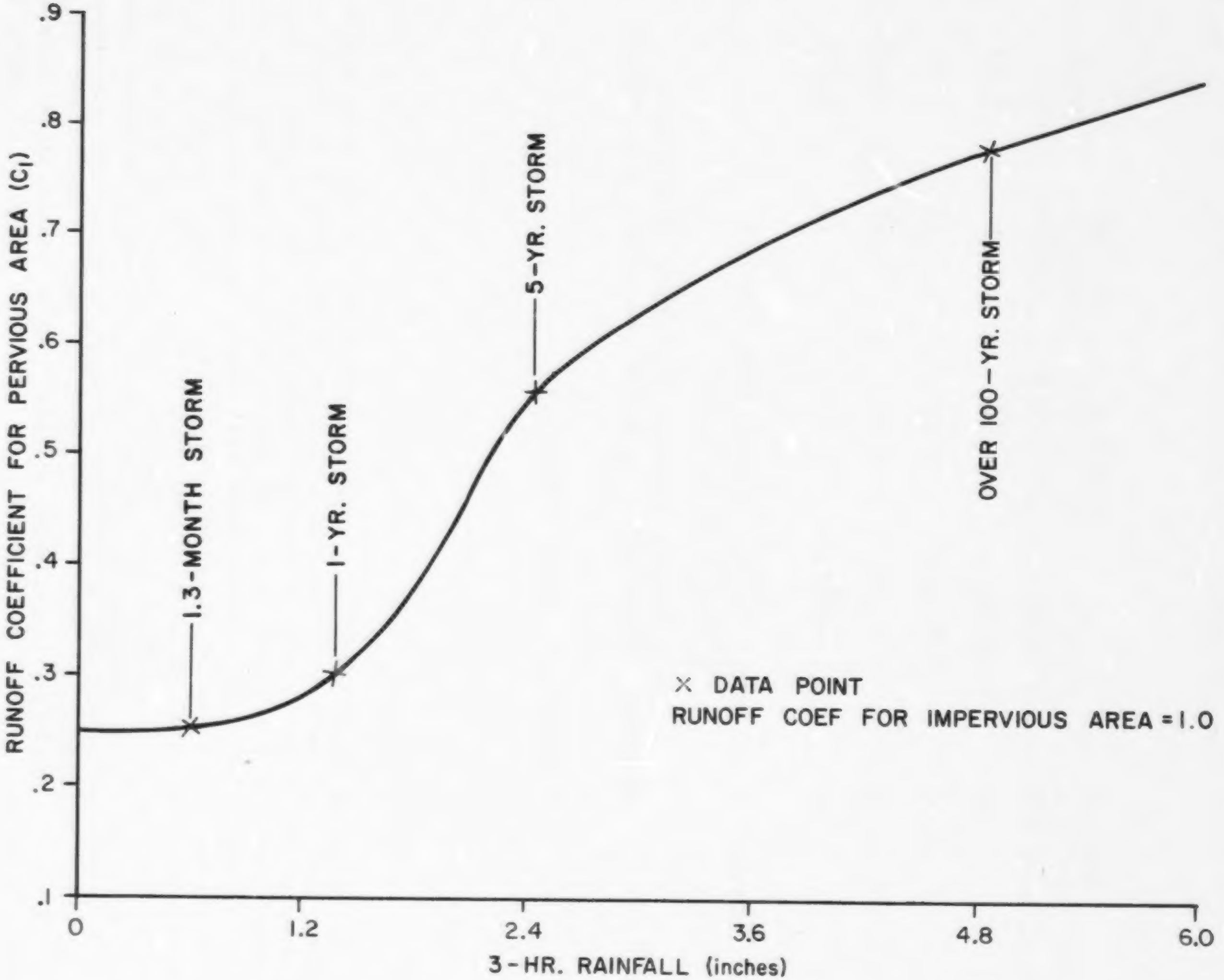


Figure E-1. STORM runoff coefficient C_1 vs. rainfall records to be simulated regardless of rainfall volume or intensity, a C_1 value of 0.25 was used for the simulation of long-term records for evaluation of overflow pollutional effect. Reducing C_1 to 0.15 would result in a nine percent reduction in the runoff volume from District A. In addition, the amount of pollutant washout from streets is independent of C_1 in analyses using STORM as previously discussed. The selected C_1 value of 0.25 should provide sufficiently consistent results when using STORM to permit valid evaluation of the effectiveness of pollution control measures. To estimate the 5-year synthetic storm runoff volume from other districts in the City, however, C_1 value of 0.55 was used.

SECTION XIV

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SECTION XV

GLOSSARY, ABBREVIATIONS, AND SYMBOLS

GLOSSARY

Advanced combined sewer system - A combined sewer system which incorporates flow routing and storage as well as flow conveyance in the design. Types of storage that could be considered include in-pipe storage and satellite storage.

Advanced pattern storms - Storms with peak rainfall occurring in the first quarter of the rainfall period.

Base sewage flow - Average rate of dry-weather flow in the sewer system after subtracting infiltration.

Combined sewer - A sewer receiving both storm runoff and municipal sewage.

Combined sewer overflow - Flow from a combined sewer in excess of the interceptor capacity that is discharged into a receiving water.

Concentration pollutograph - A graph plotting pollutant concentration per unit of time versus time.

Conventional combined sewer system - A combined sewer system without flow control devices for routing combined sewage flows. Within the context of this report, "conventional" implies the design of sewers using flows derived from a synthetic hyetograph.

Conventional separate storm and sanitary sewer system - A dual sewer system, one conveying stormwater only and the second conveying only municipal sewage. Also see storm sewer and sanitary sewer.

Depression storage - Storm water retained in surface depressions after infiltration capacity has been exceeded.

District A - A drainage district of about 265 hectares (655 acres) in the City of Elizabeth. A detailed study was made for this district to develop runoff quantity and quality characteristics using SWMM and STORM for application to the entire City.

Diurnal variation - Hourly variation in the dry-weather flow quantity and quality.

Drainage district - The area which is drained by a collection system, and, generally, to one outlet to the interceptor.

Dry-weather flow - That flow in sanitary or combined sewers that contains no direct rainwater runoff.

Evapotranspiration - Water withdrawn from soil by evaporation and plant transpiration.

First flush - The initial rainwater runoff flows which may be expected to contain high concentrations of the pollutants that were deposited in the sewer during the period antecedent to the rainfall.

5-year synthetic rainfall hyetograph - A synthetically generated hyetograph which matches at all time intervals the intensity-duration curve for a 5-year return interval prepared by the U.S. National Weather Service. In the report, it is also called "5-year synthetic storm".

5-year synthetic storm - See 5-year synthetic rainfall hyetograph.

Froude number - A numerical quantity used as an index to characterize the type of flow in a hydraulic structure that has the force of gravity (as the only force producing motion) acting in conjunction with the resisting force of inertia. It is equal to the square of a characteristic velocity (the mean, surface, or maximum velocity) of the system, divided by the product of a characteristic linear dimension, such as diameter or depth, and the gravity constant or acceleration due to gravity-all expressed in consistent units so that the combinations will be dimensionless. The number is used in open-channel flow studies or in cases in which the free surface plays an essential role in influencing motion.

Hydrograph - A graph plotting flow rates versus time.

Hyetograph - A graph plotting rainfall intensities versus time.

Infiltration - The water entering a sewer system and service connections from the ground, through such means as, but not limited to, defective pipes, pipe joints, connections, or manholes. Infiltration does not include, and is distinguished from, inflow.

In-pipe storage - Storage in sewer pipes to temporarily contain combined sewage or urban runoff. In the design of an in-pipe storage system, sewers are increased in size to provide volumetric storage in addition to flow conveyance. Regulators are installed to permit use of this storage to reduce peak flow rates reaching treatment facilities.

In-system - Within the physical confines of the area served by the collection system.

Mass rate pollutograph - A graph plotting pollutant mass loading per unit of time versus time.

Off-line storage - Storage so situated as to reduce peak flow rates and pollutant concentrations entering or flowing in interceptor and treatment facilities from the collection systems. Construction is similar to that for satellite storage.

1.3-month synthetic storm - Derived from the 5-year synthetic storm in the same manner as 1-year synthetic storm except the ratio of hourly rainfalls with 1.3 months and five years return intervals is used.

1-year synthetic storm - Derived from the 5-year synthetic storm by multiplying the 5-year synthetic hyetograph rainfall rates by the ratio of hourly rainfall rates for one year and five year return intervals.

Overflow event - Overflow occurs during a storm event. A storm event is defined by the properties of the individual rainfall events, such as duration and intensity, storm spacing, and time to empty any storage facilities. An event was defined as beginning when storage was first required and continued until the storage basin was emptied. Any number of overflows occurring during the duration of the event was considered as the same overflow event. If a rainfall produces runoff that does not exceed the sewer capacity, storage would not be utilized, and the rainfall would not register as an event. In the case of zero storage, runoff from the watershed would directly drain to the intercepting sewer, and the number of events as defined above would be the same as the number of overflow events.

Pollutograph - A graph plotting pollutional concentrations or pollutional mass per unit of time versus time.

Routing - Computing the downstream outflow hydrograph of an open channel or a sewer from known values of upstream flow, and diversions to, and withdrawals from storage.

Sanitary sewer - A sewer that carries wastewater from residences, commercial buildings, industrial plants, and institutions, together with a relatively low quantity of ground, storm, and surface waters that are not admitted intentionally.

Satellite storage - Storage in the collection sewer system for peak flow rate reduction and flow mixing. Satellite storage basins could be open, covered or tunneled structures.

STORM default values - Numerical values assigned in the STORM computer program. These values are used when they are not specified in the input data. In this report, the default values refer, in particular, to the factors which compute SS, BOD and the most probable number (MPN) of coliforms from the weight of dust and dirt washed off the street.

Storm runoff - That stormwater which flows overland and enters a sewer or a receiving water body.

Storm sewer - A sewer that carries intercepted surface runoff, street wash and other wash waters or drainage, but excludes domestic sewage and industrial wastes.

Stormwater - The water resulting from a precipitation event which may stay on the land surface, percolate into the ground, run off into a body of water, enter a storm sewer or a combined sewer, infiltrate a sanitary sewer, or evaporate.

Subarea - A subdivision of a drainage district based upon a single or one predominant land use.

Subcatchment - A subdivision of a subarea generally by topography and pipe network configuration and usually draining to one reach of pipe.

Surcharge - The flow condition occurring in closed conduits when the hydraulic grade line is above the crown of the sewer.

SWMM default values - Same meaning as STORM default values except that the values are specified in the SWMM program.

Trunk sewer - A sewer that receives many tributary lateral sewer branches and serves a large drainage area.

Weir-orifice regulator - A flow-regulating structure proposed for sewer installation. The structure consists of a weir at an angle to the main sewer line and a small branch sewer pipe to pass the normal dry-weather flow and to control the wet-weather flow rates diverted downstream. The weir provides a damming effect to store storm flow in large trunk sewers or purposely enlarged lateral sewers.

ABBREVIATIONS

Ac - acre (s)

APWA - American Public Works Assocation

BOD - 5-day biochemical oxygen demand

cfs - cubic feet per second

cfs/acre - cubic feet per second per acre

cm - centimeter (s)

DWF - Dry-weather flow

ENR - Engineering-News Record

EPA - U.S. Environmental Protection Agency

ft - feet

ft^3 - cubic feet

ft. MSL - feet above mean sea level

ft/sec^2 - feet per second square

in/hr - inch (es) per hour

kg - kilogram (s)

km - kilometer (s)

kg/day/100m - kilograms per day per 100 meters

lbs - pounds

lbs/day - pounds per day

lbs/day/100 ft - pounds per day per 100 feet

lbs/min - pounds per minute

m - meter (s)

m^3 - cubic meter (s)

m^3/min. - cubic meter (s) per minute

m^3/sec	-	cubic meter (s) per second
mg	-	milligram (s)
MG	-	million gallons
mgd	-	million gallons per day
mg/l	-	milligram (s) per liter
min.	-	minute (s)
mm/hr	-	millimeter (s) per hour
N	-	nitrogen
PO_4	-	orthophosphates
SS	-	Suspended solids
STORM	-	Storage, Treatment, Overflow, Runoff Model as released by the Hydrologic Engineering Center, U.S. Army Corps of Enginners
SWMM	-	Storm Water Management Model as released by the U.S. Environmental Protection Agency

SYMBOLS

% - percent

$ - dollar

°F - degree (s) Fahrenheit

8" - 8 inches

< - less than

> - greater than

∂ - partial differential

d/dt - first order differential with respect to time, t

d^2/dt^2 - second order differential with respect to time, t

°C - degree (s) Centigrade

C_1 - STORM runoff coefficient for pervious area

C_2 - STORM runoff coefficient for impervious area